APPLICATION OF THE KNEADING COMPACTOR AND HVEEM STABILOMETER TO BITUMINOUS CONCRETE DESIGN IN INDIANA

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by

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Joint Highway Research Project

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TO BITUMINOUS CONCRETE DESIGN IN INDIANA

TO:	K. B. Woods, Director Joint Highway Research Project	December 14, 1960
FROM:	H. L. Michael, Assistant Director Joint Highway Research Project	File: 2-4-17 Project: C-36-6Q

Attached is a final report titled "Application of the Kneading Compactor and Hveem Stabilometer to Bituminous Concrete Design in Indiana". This report has been prepared by Mr. Noel G. Gaudette, Graduate Assistant on our staff, under the direction of Professor W. H. Goetz.

The report was also used by Mr. Gaudette as his thesis in partial fulfillment of the requirement for the MSCE degree.

The purpose of the research was to study a possible Hveem design procedure which could be employed in the design of bituminous mixtures in Indiana.

The report is presented for the record.

Respectfully submitted,

Harold L. Michael

Harold L. Michael
Secretary

HLM:kms

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File: 2-4-17
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Purdue University
Lafayette, Indiana

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The guidance and encouragement provided by Professor William H. Goetz, the writer's major professor and research advisor, is gratefully acknowledged. As Research Engineer of the Joint Highway Research Project and Professor of Highway Engineering at Purdue University, Professor Goetz was helpful in outlining a procedure for uniform and orderly progress of the work.

The calibration of the kneading compactor would not have been accomplished without the time and effort offered by Professor Albert D. M. Lewis whose suggestions and assistance in instrumentation is sincerely appreciated.

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Finally, the author wishes to express a sincere thank you to the Bituminous Laboratory Staff of the Joint Highway Research Project for their assistance in obtaining and compiling the data.

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ABSTRACT

Gaudette, Noel G., Jr., M.S.C.E., Purdue University, January, 1961. Application of the Kneading Compactor and Hveem Stabilometer to Bituminous Concrete Design in Indiana. Major Professor: William H. Goetz.

This study was undertaken with the intention of indicating a suitable Hveem design procedure to be employed in the design of Indiana bituminous mixtures under heavy traffic conditions. The study had two major purposes: 1) to study the validity of using the present Hveem design method, as employed by the California Highway Department, for design of Indiana surface and binder mixtures, and 2) if some modification of the standard California procedure was indicated for Indiana surface and binder mixtures, the laboratory compaction pressure would be altered until test results for field and laboratory specimens were in substantial agreement.

In evaluating the laboratory test results presented in this study it is important to view the data as an attempt to simulate the field condition of the pavement after five years of service by varying the kneading compaction intensity, and, in a few cases, the number of kneading tamps. No attempt is made to establish the most suitable asphalt content by the Hveem design procedure since, in order to reproduce best the field properties of the mixtures, set asphalt contents and gradations are used which were established by tests on the field mixtures. The primary intention of this study was to simulate specific field conditions using a fixed mixture in the laboratory for each of four aggregate gra-

dations.

The study required sampling of bituminous pavements by taking 4-in. diameter cores and performing a laboratory correlation. Marshall tests were made on composite samples for four samplings over a five-year period and Hveem tests were made on composite and individual-course samples for the final sampling in 1959.

A mechanical kneading compactor was used for fabricating laboratory specimens with mixture properties comparable to the properties for cored specimens taken from the pavements, and samples of each mixture were also compacted with the Marshall hammer and tested for further information. The Hveem Stabilometer and Marshall stability machine were used for measuring the strength of both field and laboratory specimens. Mixture void contents were measured for field and laboratory-compacted specimens using Rice specific gravity values. Aggregate degradation was studied in a very limited way by comparing aggregate gradations and the percent of mix retained on the No. 6 sieve, for field-compacted samples and laboratory-compacted samples, to the original aggregate gradation used at the time of construction.

Tests were also made to determine the effect of kneading compaction on specimen uniformity by determining density, asphalt content, and aggregate degradation variation throughout compacted specimens. Pavement cores were recompacted using the kneading compactor and the standard compaction procedure in an effort to reproduce field density and Hveem stability values.

The test results of density and Marshall tests for composite samples from four samplings during a five-year period have shown that the traffic-compacted density and Marshall stability reached values near

maximum after one year of service. For any period after one service year, density and Marshall stability values were consistently highest at traffic-signal intersections when compared with the density and stability results at sections under more uniform traffic flow. Density and Marshall stability results showed a wide variation with respect to the wheeltrack and between-wheeltrack positions under channelized and heavy traffic, but no significant difference between positions was indicated under non-channelized traffic and lower traffic volumes.

All Hveem stability values for composite field samples were less than 35, or lower than the minimum permissible laboratory-compacted Hveem stability value used for design in California. All Hveem stability values on surface samples were less than 35 and only 20 percent of the stability values on binder samples were over 35. Void contents for pavement cores were normally low but they varied with the traffic condition and traffic volume. Extracted aggregate samples from the pavements showed degradation primarily of the plus No. 4 sieve size of coarse aggregate and a definite increase in the minus No. 200 sieve material for all tests on surface and binder samples.

The CKE and OE procedures, using extrapolations of the standard charts, indicated lower optimum asphalt contents for surface mixtures than those commonly used in Indiana and an asphalt content increase of about 1/2 percent for binder mixtures.

Variation of the kneading compaction pressure for the four mixtures used in this study showed that density increased as the compaction pressure was increased. Hveem stability values of the binder mixtures generally increased with an increase in compaction pressure and appeared to approach a maximum near maximum density. For these binder mixtures an

increase in density increased the Hveem stability. For surface mixtures, and increase in compaction pressure generally decreased the Hveem stability so that an increase in density was accompanied by a decrease in Hveem stability.

Marshall-compacted densities for each of the four mixtures were much lower than kneading-compacted densities, but Hveem stability values for Marshall-compacted specimens were well above those for pavement-core samples for each mixture.

Specimen uniformity tests for specimens cut into three layers showed that the density variation was as high as 7 pcf between top and bottom layers of a kneading-compacted specimen with the bottom of the specimen being the least dense. Asphalt contents were highest in the bottom layer and, generally, aggregate degradation was greatest in the top layer.

It is concluded from this study that, generally, the normal 500 psi compaction pressure can be used for Indiana binder mixtures, but a lower design compaction pressure should be employed for compacting more sensitive mixtures. These sensitive mixtures are usually surface mixtures, especially those surface mixtures containing high percentages of crushed limestone aggregate and a high asphalt content.

APPLICATION OF THE KNEADING COMPACTOR AND HVEEM STABILOMETER TO BITUMINOUS CONCRETE DESIGN IN INDIANA

INTRODUCTION

The necessity for extended research in the area of bituminous concrete stability has resulted in a series of investigations by several research agencies. Each study, while adding new data and contributing to a better understanding of bituminous mixture design, unfolds a series of new problems that require further investigation before definite conclusions may be stated.

In a study by Hannan (18)* it was suggested that a project be undertaken that "would give an indication of the stability values needed for satisfactory pavement performance in Indiana." Hannan also suggested that an investigation be undertaken which would establish a satisfactory compaction procedure for Indiana mixtures using the kneading compactor. These ideas fostered the investigation at hand.

The Mix Design Manual of The Asphalt Institute (49) states that "to date, the Hveem method has been used principally for the design of dense paving mixtures." The conclusions drawn from research with dense mixtures cannot be applied to the more open-type Indiana mixtures until substantial evidence is presented to affirm such conclusions. Some modification of the procedure is certainly to be expected.

An extensive program investigating the stability of bituminous

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* Numbers in parentheses correspond to references listed in the bibliography.

mixtures has been undertaken by the Joint Highway Research Project over the past decade. Goetz (13) stated in 1951 that "It is a well-recognized fact that our pavement design procedures are inadequate in general and seriously lacking in many particulars." Goetz justified this statement by adding, "...highway engineers, are faced with a serious problem - the design of pavements which are adequate to withstand the traffic expected throughout their life, while at the same time avoiding over-design and excessive costs."

In recent years bituminous mixtures have been used extensively for resurfacing concrete pavements in several states. The state of Indiana has made wide use of bituminous mixtures for this type of construction. Late investigations have demonstrated that some modification of the original mixture is necessary to avoid excessive rutting and shoving of the resurface layer. Goetz (13) comments on failure in this application by stating: "If shoving or rutting occurs in this use, we know it must be due to lack of stability in the mixture itself and not because of the movement of lower layers." In conclusion, Goetz writes: "We feel definitely that our research program on flexible pavement design will remain deficient and fall short of the goal unless our laboratory program is benefited by companion studies in the field. ...We feel that a field research program is badly needed."

The problem of rutting in bituminous concrete overlays was first investigated in Indiana in 1953. Goetz and McLaughlin (15) state: "Evidence of lack of stability of some mixtures in some service conditions has been the development of ruts in the overlay in the wheel-track areas." Their study clearly indicates, "The condition was found to be most severe at signalized intersections where the pavement and the

overlay were subjected to stresses from braking traffic and to static loads." They conclude: "...it would appear that greater densification during construction would be desirable. This matter is of extreme importance for realistic bituminous mixture design to serve severe traffic conditions."

The following excerpts from a 1957 report by Goetz and McLaughlin (15) will help to summarize the present problem which Indiana has in the design of bituminous concrete mixtures.

> The bituminous concrete most used by the state of Indiana has been included in the standard specifications without essential change as to type at least since 1934. Until about 1948, this material was used primarily as the surfacing layers of high-type flexible pavements. In this application, the performance of the material was considered to be entirely satisfactory and the stability of the mixture was not questioned.
>
> Since 1948 this bituminous mixture type has been used more and more for the resurfacing of deteriorated Portland cement concrete pavements. Also, in the period following the war and particularly in the last several years, the number and weight of heavy vehicles using Indiana highways has increased markedly. This change in the use of the material, coupled with the large increase in traffic might be expected to create problems that did not exist previously.
>
> The bituminous-concrete overlay that has been used in Indiana usually is composed of two layers: 1) a binder or leveling course which has a maximum aggregate size of three quarters to one-inch, 65 per cent coarse aggregate (material retained on the No. 6 sieve), 35 per cent fine aggregate (essentially none of which passes the No. 200 sieve), and which contains 4.5 to 5.5 per cent asphalt by weight of the mixture, and 2) a surface course which has a maximum aggregate size of $\frac{1}{2}$-inch, about 50 per cent coarse aggregate, 50 per cent fine aggregate and with about 3 per cent of the total passing the No. 200 sieve. It usually contains 6.0 to 7.0 per cent asphalt by weight of the mixture. The thickness to which each of the courses is laid is variable depending upon the condition of the road to be resurfaced, the expected traffic intensity and perhaps other factors, but a total thickness of $2\frac{1}{2}$ inches composed of $1\frac{1}{2}$ inches

of binder and 1-inch of surface is not uncommon. The asphalt cement used is a 60-70 penetration grade.

...Failures under consideration were concluded to be caused by plastic flow. The realization of these failures and their nature led the Indiana Highway Department to modify their bituminous concrete by several approaches. Asphalt content generally has been reduced in both the binder and surface layers. In some instances, the maximum aggregate size of the binder has been increased and the thickness of this layer increased with a corresponding decrease in surface-layer thickness. In other instances the ratio of coarse aggregate to fine aggregate has been increased in both the binder and the surface. Perhaps these modifications will provide the solution. There have been overlays built incorporating one or more of these modifications which appear to be performing quite satisfactorily at the present time under severe service conditions.

...In spite of this apparently satisfactory performance to date, however, it may be that an entirely new design concept is needed, if not immediately, then perhaps in the not too distant future. But accompanying a new or even modified design concept must be a better method of evaluating the mixture in the laboratory. The presently accepted strength tests for bituminous mixtures appear to be inadequate in certain cases.

For example, the mixture that rutted under traffic action until the underlying concrete showed through will meet the stability requirements of most of the standard acceptance tests. Yet under some service conditions the mix is obviously overly plastic. Highway engineers are particularly concerned with obtaining a laboratory test method which would overcome the inadequacies of the present methods...

It has been suggested that the problems presented above might be solved by use of a more densely-graded mixture. Goetz, McLaughlin and Wood (16) comment on this point in the following manner:

This bituminous concrete is based upon a design which avoids the strictly dense-graded mixture concept in favor of one which provides appreciable aggregate voids to accomodate increased percentages of asphalt. It is recognized that there are those who would suggest that the problem outlined could be solved readily by employing a more densely-graded mixture at reduced asphalt content. This is probably true, and it may well be that Indiana will be forced to abandon its present concept

> of design in order to deal with the present severe service conditions, and the even more severe conditions which we can expect in the future. It is suggested, however, that this possibility does not alleviate the basic problem that requires asphalt paving technologists to design mixtures for conditions for which our present methods of design have been shown to be inadequate.

The following paragraphs describe briefly the recent research efforts that have been made to evaluate the current Indiana bituminous mixture problems.

A series of studies relating the behavior of bituminous mixtures under repeated loads were undertaken. Wood and Goetz (54), Goetz, McLaughlin and Wood (16), and Dennis (7) contributed information on the action of bituminous concrete under repeated loads. An earlier study by Chen (6) contributed general information on bituminous concrete stability as determined in the laboratory by several common strength-testing procedures.

Chen (6) pointed out that "When a bituminous pavement is under traffic, the loads acting on the pavement will first consolidate the bituminous mixture, if it is not well compacted." This is the action intended to be reproduced by a repeated-load type of test in the laboratory. Wood and Goetz (54) theorized that two factors have brought about rutting and shoving in the wheeltrack areas:

1. Heavier wheel loads have increased the stress experienced by the overlay.

2. The accumulated permanent deformation has increased due to increased repetitions of load.

They conclude:

> The combination of these two factors has brought about a situation such that plastic deformation becomes

a significant factor. This plastic deformation is not recoverable; the lanes experiencing the majority of the wheel loads are permanently deformed. This action is especially noted in the vicinity of stop lights where the stresses in the overlay are applied slowly and over a longer period of time than on the open stretch of road.

The study by Wood and Goetz (54) presented other valuable information to be considered in the design of bituminous mixtures. They comment:

> Field performance indicates that extreme conditions of loading for a bituminous layer occur during summer climatic conditions under a stationary load or at locations where heavy trucks with high tire pressures undergo slowing or stopping movements. The plastic component of deformation under these conditions assumes major importance. The proper mixture design should minimize this component.
>
> While it is important that a mixture should have sufficient stability under the above conditions, it must be remembered that winter temperatures impose a situation upon the mixture such that it acts more like an elastic body. Under low temperatures, the mixture could become quite brittle.

Dennis (7) conducted his study of repeated-loading under the premise that "Since rutting is usually accompanied by a change in density and amount of voids filled with asphalt, there should be a relationship between rutting and stability."

Currently the triaxial test is being employed in research studies in an attempt to minimize the present problems with Indiana bituminous mixtures. Contributions have been offered by Oppenlander and Goetz (37), Schaub (42), and by Hannan (18) who used the Hveem Stabilometer. The equipment and manner of the strength tests for these studies differed greatly but each investigation has very definitely added to our knowledge of bituminous mixture stability.

The present trend is toward continued use of the triaxial and re-

a significant factor. This plastic deformation is very noticeable; the beads experiencing the majority of the wheel loads are permanently deformed. This problem is especially evident in the vicinity of main landing gear. [illegible] a longer period of time [illegible].

The study by [illegible] (11) [illegible] valuable information to be considered in the design of [illegible]. [illegible]

"[illegible] of [illegible] in [illegible] [illegible] [illegible] of [illegible] [illegible]."

"[illegible] [illegible] [illegible] [illegible] quite [illegible]."

[illegible] (12) [illegible] that "[illegible] is [illegible] the amount of [illegible] between [illegible] and [illegible]."

Currently the [illegible] in an attempt to [illegible] [illegible] [illegible] [illegible] (13), and by [illegible] (14) [illegible] [illegible] of the [illegible] [illegible] [illegible] of [illegible] [illegible].

The [illegible] [illegible]

peated-load procedures as research techniques, with an increase in the use of the Hveem Stabilometer for design purposes since the use of the Hveem design method by the Indiana Highway Department. It is hoped that the Hveem method will provide a rational design. Hennes (19) writes: "The regional popularity of this method stems from the fact that the accompanying methodology takes into account all of the principal factors which have been thought to be influential in affecting pavement performance."

Reproduction of conditions in the field by a laboratory compaction method has long been a serious problem and the advent and growing use of the kneading compactor has greatly overcome these difficulties. Hennes (19) comments on this as follows:

> The structural arrangement of aggregate particles evades direct measurement. One recourse in this dilema is to seek to achieve the same structural effects in our test specimen as those prevailing in the field, by careful laboratory simulation of field construction practices. ...Much of the attention of the Triaxial Institute for structural pavement design has been devoted to the development of a satisfactory kneading compactor which would shove, rather than pound, aggregate particles into more compact arrangements.

The versatility of the kneading compactor, and its elimination of a large part of the personal error involved in compacting specimens, has made its use very desirable. Schaub (42) employed the kneading compactor to obtain rational triaxial specimens of uniform density and asphalt content. Hannan (18) did not use the kneading compactor for the bulk of his work; however, his study is the most informative to date on testing Indiana bituminous mixtures with the Hveem Stabilometer.

The compaction and testing procedure generally followed in the study at hand are presented in the California Materials Manual (47).

REVIEW OF LITERATURE

A review of the development and reasoning behind the use of the Hveem design method is the primary function of this section. In the INTRODUCTION of this report many studies have already been reviewed in an effort to present the basic problem and purpose of this investigation.

In order to present the common techniques of sampling, compacting and testing followed in this investigation, the review of literature is composed of five topics:

1. Stability of bituminous mixtures
2. Highway performance studies
3. Compaction of bituminous concrete
4. Soils investigations
5. Marshall and triaxial versus Hveem

Stability of Bituminous Mixtures

Neppe (34) in his literature summary on mechanical stability of bituminous mixtures makes several pertinent comments. He states that the ability of an asphalt pavement to carry traffic loads is primarily dependent upon the mineral aggregate. This ability depends on internal friction and on the mechanical arrangement of interlocking of the individual particles of the mass, which are greatly affected by the degree of compaction, particle shape or angularity and surface texture in addition to the aggregate grading.

Neppe's report summarizes ideas of Stanton and Hveem who believed "that the surface character of the mineral aggregate is the most im-

portant single quality affecting the stability of a bituminous pavement and advocate the use of rough stone to maintain as much friction as possible between the particles."

Investigations(21, 24, 44) have shown that aggregates are eventually crushed in service to a size distribution approaching a maximum density. Stanton and Hveem (44) do not suggest using such a gradation for design since they believe this will lead to critical conditions in which the voids tend to become too easily overfilled and the mixture unstabilized.

Hveem (21) writes: "It was a little disturbing to discover that some of the most unconventional and irregular grading curves were identified with the most successful roads, while in several failures, the gradings complied very nicely with orthodox ideas as represented by Fuller's curve." Neppe (34) states that Endersby substantiates this by pointing out that, when voids decrease as a result of compaction, a stable pavement may lose its stability in practice because of excess asphalt. Stanton and Hveem (44) do not believe that low void volume is an essential measure of mixture quality. Hveem (21) emphasizes that equilibrium characteristics depend upon the conditions at contact points between discrete particles.

There is agreement that the most suitable content of bitumen in a mixture is the largest amount that can be tolerated by the aggregate without developing instability. However, Neppe (34) concludes that "the whole field of bituminous pavement design is covered with empiricism and controversy."

Work by Herrin and Goetz (20) indicates that the stability of a

mixture may be improved by use of a crushed-stone fine aggregate instead of natural sand. Their results also demonstrate the strength of a mixture is influenced more by the fine aggregate than by the variations in shape of the coarse aggregate.

Herrin and Goetz (20) conclude that grading of the aggregate is an important factor affecting the strength of bituminous-aggregate mixtures and has more effect on strength than does the shape of the aggregate.

An interesting comment on stability is provided by Lottman and Goetz (26): "Since in many cases as little as 25 per cent crusher dust produced a significant increase in mixture strength for those mixtures tested, it appears that this means of providing stability increase should be given consideration in those areas where mixtures produced with natural sands lack sufficient stability."

Griffith and Kallas (17) affirmed these effects of fine aggregate on stability. They added that the fine aggregate type had considerable effect on the aggregate voids characteristics as well as on stability of mixtures.

Highway Performance Studies

More and more, state highway departments are incorporating performance studies into their bituminous pavement design programs. Although the information obtained from such studies is generally quite variable, it is of extreme value, especially when studies are conducted on pavements for a period of several years.

Field performance studies provide information on the actual properties of the in-service pavement. These results are extremely valuable in preparing laboratory specimens since laboratory test results

mixture may be increased by use of a crushed stone fine aggregate instead of natural sand. Investigators also demonstrated the strength of a mix is influenced more by the fine aggregate than by the variations in shape of the coarse aggregate.

Neville and Kenny ([illegible]) conclude that grading of the aggregate is an important factor affecting the strength of bituminous-aggregate mixtures and has more effect on strength than does the shape of the aggregate.

An interesting comment on stability is provided by [illegible] and [illegible] ([illegible]). "Since increasing [illegible] by 15 per cent [illegible] produced a significant increase in [illegible] strength [illegible], it [illegible] that [illegible] would be [illegible] in those areas where [illegible] with natural sands [illegible] conditions."

[illegible] and [illegible] ([illegible]) [illegible] of stability. They [illegible] that [illegible] effect of the aggregate [illegible] of [illegible].

[illegible]

[illegible]

are valid only if the specimen is representative of field conditions.

Goetz (12) writes that traffic is one of the three factors difficult to evaluate by laboratory or field tests alone. He adds: "However, laboratory or field tests, when used in conjunction with field performance surveys, can provide excellent design information." Goetz concludes: "Pavement performance surveys have and will continue to provide the necessary correlation between the laboratory and the field."

Vallerga (51) defines instability as deformation of the pavement under traffic loads and presents a table to categorize three major failure types and the causes of each. Table 1 is extracted from this table.

Vallerga proposes three general rules to be observed in the design of an asphalt mixture:

1. Select a good quality, relatively hard, hydrophobic, rough-textured aggregate of any desired grading to meet requirements of workability, permeability, pavement texture, and economy.

2. Choose asphalt consistency as soft as possible to insure good workability and long life but still hard enough to provide adequate tensile strength and resistance to water for the grading selected.

3. Use as much asphalt as the mix will tolerate without loss of stability.

Vallerga (52) has stated that road building will always be an art in which experience and records of performance will always influence engineering decisions.

Neppe (34) lists "deformation or flow caused primarily by the load carried" as the foremost type of failure in bituminous pavements. He emphasizes that the effects of braking and acceleration are considerably more serious than free-running loads.

Stevens (48) presents several comments of interest relative to the failure of bituminous pavements under traffic:

> The load bearing capacity of an asphalt mix is dependent on both the asphalt and the mineral matter. But asphalt is a material which will move and flow and adjust itself, while mineral matter or aggregate resists movement until overloaded.
>
> ...Sudden application and release of a load on paving asphalt has little effect on it as the rate of this flow is quite slow, but, if a load rests for some time on a piece of paving asphalt, it will sink some distance into the asphalt.
>
> For this reason, even a pavement in which the properties of the asphalt are very pronounced will continue to carry rapid-moving traffic because the paving asphalt resists the shock loading it receives under these circumstances.
>
> However, if traffic comes to a halt on such a pavement, the slow flow properties of the asphalt become evident and the mix shoves up in ridges.

Compaction of Bituminous Concrete

It is generally agreed that the manner in which a specimen is formed in the laboratory is of utmost importance if the stability measured is to be meaningful. Goetz (14) substantiates this by stating: "The most important phase of the present design problem is not how to test a laboratory specimen, but how to form a laboratory specimen that represents the mixture as it is used in service."

Monismith and Vallerga (32) write:

> In the design of asphaltic paving mixtures by means of laboratory tests, the object of laboratory compaction procedures should be to produce specimens which are representative of materials of similar composition compacted by construction equipment and traffic in the field.

Smith (43) writes:

> Precise testing of stability properties of soils, aggregates, and bituminous mixtures used in flexible

Brewer [illegible] presents several comments of interest relative to the failure of bituminous pavements under traffic:

> The load bearing capacity of an asphalt mix is dependent on both the asphalt and the mineral matter. The asphalt is a material which will creep and flow under load itself, while mineral matter or aggregate resists movement under load.

> [illegible] application and release of a load to the asphalt has little effect [illegible] if the rate of [illegible] is quite slow. But, if a load [illegible] for some time on a [illegible] of paving asphalt, it will [illegible] distance into the asphalt.

> For this reason, [illegible] pavement [illegible] portion of the asphalt [illegible] to carry rapid-moving traffic [illegible] asphalt [illegible] construction.

> However, if [illegible] traffic [illegible] the asphalt [illegible] and the [illegible].

<u>Comparison of [illegible]</u>

It is generally [illegible] that the [illegible] in field and in the laboratory [illegible] is to be [illegible] the most important phase of [illegible] a laboratory specimen, but [illegible] present [illegible] as [illegible] (Fig. [illegible]).

Monismith and [illegible] (19[illegible]) state:

> In the design of asphalt paving mixtures [illegible] of laboratory [illegible] specimens [illegible] as in construction equipment [illegible] in the field.

[illegible] (19[illegible]) writes:

> Present methods of compacting [illegible] static, gyratory, and [illegible] kneading [illegible]

Table 1

Table for Improving Stability Performance of Asphaltic Pavements (a)

Type of Failure	Causes	Function of
Instability	Low interparticle friction	Particle surface texture Quality of asphalt (excess) Particle contact pressure (compaction)
	Low liquid friction or mass viscosity	Gradation or surface area (points of contact) Types and amount of asphalt Density
	Lack of inertia resistance	Speed of vehicle Weight of vehicle Mass of pavement affected

(a) Vallerga (51).

> pavement construction involves the preparation of test specimens of densities and gradations closely approximating those that can be obtained in actual construction practice.

Smith adds:

> Density may be a misleading criterion for the judging of the stability of a mix. ...Field observations showed that certain asphaltic surfaces possessed adequate stability for several months or more after construction, but eventually showed some shoving distress. Investigation of this situation revealed that some mixes of adequate internal friction and cohesion at the time of construction became over-lubricated due to densification under traffic. As the result of this densification, the stability properties of the mix as measured by the triaxial test were greatly reduced.

Monismith and Vallerga (32) present similar conclusions: "From experience, however, it has been found that densification of some asphaltic mixtures in the road by traffic over a period of time has actually resulted in a decrease in stability."

In 1957 the Association of Asphalt Paving Technologists presented a symposium on compaction. The following excerpts are from a paper presented by Nevitt (35) at that meeting:

> Compaction is an energy-consuming process in which forces act to produce a definite result. Complexities or differences in this result, often large, arise from wide variation in the forces, in their magnitude and duration, and in their direction, along with similar differences in the possible resistance effects.
>
> ...Consolidation or road compaction utilize relatively low intensity forces repeatedly applied at varied rates but without impact and with alternating horizontal components developed from all directions.
>
> ...Direct compression required high force intensity to produce any density, results in negligible particle orientation, and also causes excessive degradation with many aggregates.
>
> ...Impact effects are accomplished by high stress intensities, correspondingly high inertia and flow resistances, and some degradation.

...Vibration superficially appears a quite different compacting agency than traffic.

...No standardized test generally used today can be set up to assume such duplication, since each mix may require a different laboratory compactive effort to produce the same effects as consolidation or other compaction due to the inherent difference in mix characteristics. A laboratory compaction method which will do this must duplicate the elements of traffic action with reasonable exactness.

The Marshall method of compaction has been used for extensive correlation studies. The density obtained is usually that which is produced by the construction equipment according to Marshall (27). Dillard (8) affirms this, and adds that construction densities are quite variable since the time and temperature of rolling are variable.

In 1948 the Triaxial Institute was formed to develop a scientific, yet rapid, testing procedure. It was readily realized that no present compaction method was satisfactory and work was begun on a new compactor which is commonly referred to as a kneading compactor. In reporting on the compactor, Endersby (9) comments that the particle orientation obtained approaches that obtained in a pavement under traffic compaction. He presented three conclusions:

1. Results differ radically in some cases with method of compaction regardless of density achieved.

2. That an over-all correlation cannot be obtained because the same relation does not hold good for all aggregates.

3. That only kneading or rolling methods are likely to reproduce road conditions.

Hveem and Davis (24) have stated that experience indicates a kneading-type compaction is capable of producing an amount of degradation of the material comparable with that which occurs in a pavement. Smith (43) affirms this by the statement: "Kneading-type compaction is known

...Fifty-[illegible] [illegible] using different [illegible] [illegible].

...[illegible] [illegible] that [illegible], such tests [illegible] be set up to [illegible] such [illegible], since each [illegible] a different laboratory [illegible] effects [illegible] the same effects [illegible] due to the [illegible] in data characteristics. A laboratory [illegible] should [illegible] the [illegible] of [illegible] [illegible] computer.

The original method of computation has been used for [illegible] statistics. The [illegible] [illegible] [illegible] [illegible] [illegible] since the time and [illegible] [illegible].

In 1958 the [illegible] [illegible] rapid, testing procedure. [illegible] computation method was satisfactory, [illegible] which is commonly referred [illegible] the computer. [illegible] [illegible] approaches that [illegible] [illegible] to proposed three conclusions:

1. [illegible] method of computation [illegible].

2. That so long [illegible] between the same [illegible] approximate.

3. That only [illegible] is by [illegible] conditions.

[illegible] [illegible] that [illegible] different [illegible] of the material [illegible] [illegible].

to yield specimens approximating closely the particle orientation and stability properties obtained in actual field construction." He continues: "For bituminous mixes possessing less than about 5 per cent voids and for many soils and aggregates, kneading-type compaction has been the only suitable method for preparing test specimens which exhibit intergranular friction characteristics equivalent to observed field behavior."

A study by Vallerga (50) of comparative Hveem Stabilometer values for three common compaction methods indicated that only kneading compaction definitely correlates with pavement performance. Obviously, some phenomena, other than density, influences the results. This is most probably orientation of the particles.

Endersby and Vallerga (10) state:

> In running down possible differences between laboratory and field compaction the first fairly obvious point is that the pavement as laid down has no rigid mold around it, which is a major factor in both compaction and particle arrangements in laboratory specimens. This situation can be changed by working particles away from the mold, by making the mold flexible or yielding, by shaking them loose from the mold, or by taking the mold away altogether. The chief difference between any kind of mold compaction, and roller or pneumatic-tired compaction in the field, is that adjacent to the roller or tire the particles can move laterally or longitudinally with considerable freedom; and with a fair amount of freedom vertically also.
>
> ...The roller and traffic condition can be approached closely if the pressure is applied through a loaded area smaller in diameter than the mold and the material is pushed around and around the mold to any desired degree.

Endersby and Vallerga (10) conclude that kneading compaction produces stability curves which are more sensitive to asphalt content than other methods of compaction. Higher magnitudes of stability are commonly obtained at low asphalt contents and lower magnitudes at high asphalt

contents. The versatility of the machine is also a pronounced feature. The state of California uses a tamping pressure of 500 psi to obtain densities a little on the light side of traffic compaction and the state of Washington uses a tamping pressure of 350 psi to obtain densities a little on the heavy side of traffic compaction (10). This implies that the compaction of laboratory specimens may be accomplished with the kneading compactor for a wide compaction pressure range to agree with particular design data established by traffic variations. Another feature of the versatility of the compactor is that by increasing the compactive effort it is believed that mixes which will drop in stability with additional compaction can be detected.

The machine will compact specimens up to 6 inches in diameter and 12 inches high. Schaub (42) used a kneading compactor for compacting rational triaxial specimens of uniform density and asphalt content.

The Texas Highway Department has made extensive use of a kneading compactor that, unlike the machine built by the Triaxial Institute, operates on a gyratory principle. According to Philippi (39), the compactor has the ability to produce a specimen with density and degradation characteristics common to bituminous concrete pavements in Texas.

McRae (30) describes a mechanized type of gyratory compactor developed by the Army Corps of Engineers. This machine has proven to be satisfactory to produce densities corresponding to those produced under high-pressure-tire channelized traffic of heavy military airplanes. Degradation appeared to be the same as in the pavement for minus No. 200 sieve material according to McRae and McDaniel (31), but breakdown was considerably higher for larger aggregates in the actual pavement.

Properties of specimens compacted by gyratory methods are closer

to specimen properties obtained with the Triaxial Institute kneading compactor than any other common type of compaction (35).

In molding laboratory specimens to compare with pavement samples there is always the problem of how close the property characteristics should be in order to be satisfactory. It seems reasonable that if we duplicate the compaction procedure to a point where equal stability is obtained at the same density, the specimens have essentially the same internal structure.

The aim in California has been to prepare specimens representing the pavement at the age of one year because it has been found that, usually, the additional compaction of the traffic will bring out any instability within that one year period.

In selecting a design criteria, there are three principal conditions of pavement stability which must be considered. McLeod (29) lists these three conditions as follows:

1. Stability under stationary loads.

2. Stability under loads moving at a relatively high and reasonably uniform rate of speed.

3. Stability under the braking and accelerating stresses of traffic.

The most critical condition is the first of these conditions for most pavements. The third condition is second in severity and the second condition is seldomly, if ever, the critical factor in design.

Braking stresses shove the pavement forward and accelerating stresses shove the pavement backward. McLeod (29) states that braking stresses are most probably more severe than accelerating stresses from their very nature. McLeod concludes that the resistance to braking stresses increases with an increase in the pavement thickness. This implies that

for a given braking stress, a thin pavement must be designed to have a higher minimum stability than a thick pavement. McLeod nullifies the use of the braking stress criteria for design by stating that the tendency of the vertical load to squeeze the pavement out from under the tire seems to present a more critical condition than the braking stress itself.

Nevitt (35) emphasized that the critical condition of a pavement is after maximum consolidation has been achieved by traffic compaction. He points out that the final density increase above the construction density values may be small but the compaction effort is much higher as a result of the many traffic coverages during service. Nevitt believed that the increase of compaction effort by traffic greatly affects the pavement stability and in conclusion he states:

> To compare compaction effects safely or to determine if they simulate consolidation in traffic, highly compacted specimens must be resorted to.

Nevitt adds that results from several projects cannot be averaged together if a reliable design criteria is to be established. He reasons that the need to compare specimens under conditions of maximum consolidation or compaction is primarily the result of orientation effects.

It is believed that the most critical compaction situation is that of maximum consolidation since the most serious failure can correspond to this condition. For standing loads, bituminous mixtures have but little more capacity to support loads than the same thicknesses of untreated crushed stone. For rapidly moving traffic there is a tremendous increase in this resistance. However, traffic lanes are not parking strips; and shearing strength under static conditions cannot be directly related to flexible pavement performance under traffic.

for a given loading system, a thin pavement must be designed to have a higher minimum stability than a thick pavement. Nelson believes that one of the bending stress criteria for damage to paving [illegible] that the [illegible] strength of the [illegible] tends to prevent the pavement from [illegible] under the tire acts to prevent a more [illegible] than the [illegible] itself.

Marais (19[illegible]) summarizing [illegible] density of a pavement is often maximum [illegible] in temperature. He points out that the final [illegible] the [illegible] of density values may be [illegible] as a result of [illegible], [illegible] than the [illegible] of [illegible] the pavement [illegible]

In general [illegible] should be [illegible] in terms of their [illegible] [illegible] Highly [illegible]

Hveem [illegible] together [illegible] that [illegible]

It is believed that [illegible] of maximum [illegible] to this condition. The [illegible] little more [illegible] support [illegible] increase in this [illegible] and [illegible] ly related to [illegible] pavement performance under traffic.

Voids are not considered as being too significant in the Hveem design method as long as they are not filled. When the voids become filled there is a rapid drop in the mix stability. It is generally accepted that the use of a voids criterion for establishing stability is not a reliable procedure, however, its use for controlling durability is not argued.

Pavement instability failures caused by the excess compaction occurring under heavy channelized traffic has brought about a reduction of 20 per cent of the bitumen normally used under such traffic conditions. This is in anticipation of not allowing a zero voids condition to develop.

The above discussion justifies the avoidance of the use of voids in stability design and the working out of a design procedure based on kneading-compaction density and a good stability test.

Soils Investigations

Several studies have been conducted with soils which are helpful in explaining some of the results obtained with bituminous mixtures. A similar effort to reproduce field conditions has been undertaken for soils as for bituminous mixtures. Kneading compaction seems to be the most desirable type of compaction and several devices have been used including the kneading compactor developed by the Triaxial Institute and a miniature kneading compaction device which has been developed at Harvard University.

Seed and Monismith (4) report that in their work with kneading compaction that for a given degree of saturation in a soil, stability always increases with increasing density. Therefore, design of pavement

Folds are not considered as being too significant in the knee de- sign method as long as there are no [illegible]. When the [illegible] become [illegible] there is a rapid drop in the end stability. It is generally accepted that the use of a [illegible] for [illegible] is [illegible] a reasonable proportion, however, the [illegible] stability) is not [illegible].

[illegible] instability [illegible] caused [illegible] occurring under [illegible] of 50 percent of the [illegible] 1952. This is an [illegible] of the [illegible] to develop.

[illegible] the [illegible] of [illegible] stability [illegible] and the [illegible] bending-compression [illegible] and a [illegible].

Flange Considerations

[illegible] explaining some of the [illegible] A similar [illegible] as [illegible] most [illegible] including the [illegible] and a [illegible] [illegible].

[illegible] and [illegible] that for a given degree of [illegible] as [illegible] increases with increasing [illegible]. Therefore, [illegible]

subgrades is generally for highest practical density. However, they state that there is considerable evidence available, both in the field and laboratory, to indicate that maximum strength and maximum density of a soil are not necessarily attained at the same time. It is only for partially saturated soils that increased density may have a deleterious effect on stability. Foster (11) explains this phenomena as follows:

> The decrease in strength above certain conditions of density in both the laboratory and field tests is believed caused by the development of pressure in the void phase of the soil structure. As long as the combination of moisture and density is such that no significant pore pressures develop, increases in strength occur with increases in density. When pore pressures develop, further increases in density produce decreases in strength.

Foster (11) further states:

> The results of unconfined compressive strength tests will also show a decrease in strength with increase in density at high densities. Triaxial test results will show this behavior if the deviator stress at a low percentage of strain is used; they do not show this behavior if the maximum or ultimate value of the deviator stress is used.

Thus, the results depend on the stability criterion which is adopted. For large permissible strains it would be expected that stability would increase with density. For measurements made by the Hveem Stabilometer, the permissible strain is low and a density increase at a given water content may cause a decrease in stability depending on the range of densities and the water content of the soil involved.

The effects of traffic on pavement soils as compared to the effects on the bituminous surface is believed to be one of the main indications of the different functions served by the two materials. It is generally accepted that soils are not usually densified or activated by traffic to

compaction is generally for highest practical density. However, there will then exist in considerable scatter--particularly, both in the field and laboratory, to indicate that maximum strength and maximum density of a soil are not necessarily attained at the same time. It is only for partially saturated soils that increased density may have a detrimental effect on stability. Foster (11) explains this phenomenon as follows:

> The decrease in stability above certain conditions of density in both the saturated and field cases is caused by the development of pressure in the pore water of the soil specimens. At high moisture contents and densities, as soon as load is applied, pore pressures develop, [illegible] pressure is [illegible] increases in density [illegible].

Bennet (11) further states:

> The results of unconfined compression tests will also show a decrease in strength with increase in density at high moisture [illegible] these tests indicate that the [illegible] in the [illegible] strength for a given [illegible] if the specimens [illegible] is [illegible].

Thus, the results [illegible] considered. For large [illegible] that stability would [illegible] of the [illegible] in the [illegible], the [illegible] given [illegible] may become a [illegible] in the range of densities and [illegible] of the soil [illegible].

The effects of [illegible] on [illegible] to the effects on the [illegible] surface is [illegible] of the [illegible] of the [illegible]. It is [illegible] that soils are not [illegible] by [illegible] to

any great extent because of their position under the bituminous surface in a pavement.

Marshall and Triaxial Versus Hveem

There are numerous methods available for measuring the strength of bituminous mixtures, but not all of these methods have found wide acceptance in bituminous design work. The Marshall method is employed by many state highway departments and the triaxial method has found wide acceptance as a research tool. Since the Hveem Stabilometer is believed by many engineers to be a type of "closed system" triaxial test, and since it is used in Indiana both by the State Highway Department to develop design information and by the Joint Highway Research Project as a research tool, the advantages and disadvantages of the Hveem method are compared with the triaxial and Marshall design methods.

Of the many mechanical stability tests available, few make any attempt to simulate conditions imposed by traffic on a road surface. Most tests consist of applying some kind of stress to a test piece and measuring the resulting deformation. The manner in which this stress is applied is of primary importance. Stevens (48) states:

> A test method which subjects a specimen to rapid loading will bring into play the full viscous resistance of the asphalt to rapid deformation. Such a test method will correspond to the rapid-moving traffic load condition in the field, but on the other hand it will not give a true indication of the ability of the mix to withstand slow or static loading.

It is commonly accepted that bituminous mixtures tested at 140°F represent a critical field condition. Stevens (48) writes: "If a laboratory speciemen is tested at room temperatures there will be a tendency for the asphalt to influence the stability results by giving higher

[illegible] great extent because of [illegible] creating [illegible] bituminous surface [illegible].

Marshall and Triaxial Design Tests

There are numerous methods available for measuring the strength of bituminous mixtures, but not all of these methods have found wide acceptance in bituminous design work. The Marshall method is employed by many state highway departments and the triaxial method has found wide acceptance [illegible] [illegible] [illegible] is to be [illegible] by many [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] is used [illegible] by the State Highway Department [illegible] [illegible] design [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] and Marshall [illegible] [illegible].

Of the [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] bituminous [illegible] [illegible] [illegible] [illegible] [illegible] surfaces. [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible].

> [illegible].

It is [illegible].

readings than would be characteristic of a road during hot weather."

In judging the ability of a paving mix to be stable within itself, Stevens believed that the "principle issues are how much of the load will it transmit vertically to the course below it, and how much of the load will it transmit laterally, thus tending to shoving, rutting and upheaving."

In conclusion Stevens comments that once a standard has been set for a laboratory test by testing it against several known good and poor pavements, "a relatively dependable tool has been developed for that general class of conditions."

However, even after a suitable test method has been adopted there is the problem of deciding whether to design for the moving load condition, or for the more severe static-load condition. In the latter case the viscous resistance of the asphalt could be considered as an added factor of safety although it is believed to be quite small in a static-load situation.

The Marshall test uses a maximum load design criteria with compensation for the deformation of the sample by a "flow" value. The test is termed a reverse compression test in which a lateral load is applied to a specimen that is free to flow axially.

Endersby and Vallerga (10) report that the flow curve when inverted would parallel the Stabilometer curve. This implies that the flow value is probably more indicative of stability than the measured load on a test specimen. It is generally believed that the Marshall strength value is a better measure of cohesion than of stability of a mixture. It has been shown that the Marshall stability curve closely resembles the Hveem Cohesiometer curve (10, 51). Thus, density and cohesion are

more closely related than density and stability. Endersby and Vallerga (10) have correlated Marshall and Hveem stability measures by using specimens compacted with a kneading compactor for both tests. The use of a single compaction method assists materially in evaluating results obtained by different test methods.

The Marshall method will consistently show a higher optimum asphalt content than the Hveem method indicating that the Marshall Test is more a measure of cohesion than the Hveem stability test. The value obtained for optimum asphalt content by the Marshall Test is commonly near the optimum obtained by the Hveem Cohesiometer. The practical use of a higher asphalt content results in a higher tensile strength but problems of excess flexibility in the pavement develop. Regarding this Stanton and Hveem (44) write:

> The fact that mixtures of very low tensile strength can and do remain smooth under traffic, and also that mixtures of quite high tensile strength have been known to become waved and rutted, is proof that high tensile strength is not essential for resistance to the distorting effects of vehicles.

The triaxial test is conducted at room temperature and under a static loading. To simulate field conditions this seems to be an erroneous approach, but such limitations are necessary to measure the desired test properties of cohesion and internal angle of friction. Since the triaxial test allows means for carefully controlling and measuring a number of variables, it has been most attractive to those interested in theoretical studies and in investigations of basic principles. The use of a rational test specimen is a strong argument in favor of this method.

The primary disadvantage with the test is the time involved.

Smith (43) reports a period of approximately two hours as being required to test each specimen. For routine laboratory testing this is not at all feasible.

The importance of the triaxial test is presented by Neppe (34) as follows:

> It is universally agreed that triaxial methods hold the greatest promise for the development of rational design data. Owing to their complexity, however, efforts are at present being directed at simplification without destroying the fundamental character of the tests. The aim is to establish reliable data, within reasonable tolerances, which can adequately be correlated with service behaviour and which may then be applied with confidence, empirically or otherwise, to the straight-forward solution of outstanding problems.

A west coast organization known as "The Triaxial Institute" studied the possibilities of developing, standardizing and promoting the principles involved in the triaxial test procedure. Vallerga (53) reports that the Institute went on record, as of 1953, as endorsing and recommending the use of the Hveem Stabilometer for strength testing of bituminous mixtures, provided that test specimens are prepared by an approved kneading-type compaction. Thus, the kneading compactor is an integral part of the Hveem method and this manner of specimen preparation is implied when speaking of Stabilometer specimens.

The test measures the total shearing resistance of the specimen, which is a fundamental physical property. All physical factors which are considered to be influential in affecting pavement performance are taken into account. As performed, the test is influenced only slightly by the portion of the total resistance due to cohesion.

Zube (55) reports that stability values fall very rapidly beyond a certain limit which is believed to be the result of complete saturat-

ion of the aggregate voids with asphalt. Specifying a lower asphalt content has been good practice to allow some latitude for construction variations. Most currently used design methods tend to recommend the use of too much asphalt, as indicated by actual field performance.

The Hveem method generally necessitates consideration of an optimum asphalt content obtained in the Hveem Cohesiometer test which will invariably be higher than the Stabilometer optimum. However, the previous discussion presents substantial evidence that cohesion is not a serious problem in the range we are working in and more suitable asphalt contents will likely be indicated by the Stabilometer test.

Hveem and Davis (24) point out the speed and simplicity of the test method are its greatest advantages. The test also readily adopts itself to testing cored specimens, providing a most satisfactory correlation arrangement.

The test utilizes many of the principles of the triaxial test with its widest deviation being the sample size. The specimen height used in the Stabilometer has been selected to reasonably correspond to a typical thickness of bituminous surfacing. Concerning specimen heights, McCarty (28) presented the following criteria:

> Thus, while it is not correct to base the design of comparatively thick base courses on results from a Hveem Stabilometer test on the small Hveem specimen without applying a height correction derived experimentally or from theory, if possible, for the relatively great difference in structural strength, neither is it correct to apply uncorrected results from the test on a tall specimen in the design of thin bituminous-surface courses.

Hveem and Davis (24) are of the opinion that there is no reason to believe that a series of materials in a range of high to low stability would differ any in relative classification whether tested in the Sta-

ion of the aggregate which with asphalt. Identifying a lower minimum content has been good practice to allow some latitude for compensating variations. With currently used design methods there is [illegible] the use of too much asphalt, as indicated by actual field performance.

The Hveem method generally [illegible] of an optimum asphalt content [illegible] in the [illegible]. The [illegible] lower value will invariably be higher than the Marshall [illegible] optimum. However, the previous discussion presents substantial evidence that bleeding is not a serious problem in the range of [illegible] asphalt contents [illegible] estimated by the Marshall [illegible] test.

Hveem and Davis ([illegible]) [illegible] the speed and simplicity of the test method [illegible] advantages. The [illegible] readily adapts itself to [illegible] control [illegible], providing a [illegible] relation [illegible].

The [illegible] of the [illegible] of the [illegible] test with the [illegible]. The [illegible] in the Stabilometer has been [illegible] to a [illegible] thickness of [illegible]. McLeod ([illegible]) presented the following [illegible]:

> Thus, while it [illegible] design of [illegible] that [illegible] from a Hveem Stabilometer [illegible] of [illegible] out [illegible] a [illegible] or [illegible], if possible, for the [illegible] difference in [illegible] strength [illegible] to apply [illegible] results from the test of a [illegible] in the design of this [illegible] surface [illegible].

Hveem and Davis ([illegible]) [illegible] of the [illegible] that there is no reason to believe that a [illegible] of materials in a range of [illegible] the stability would differ [illegible] relation [illegible] whether [illegible] to the [illegible]-

bilometer or a triaxial device using standard test sizes for each method. This idea is somewhat verified by Hannan (18) in the following conclusion:

> Stabilometer tests on the open-graded mixture A gave strain measurements very near to those required to develop the mix's maximum shearing resistance in a rational triaxial compression test of the same mixture conducted at confining pressures similar to those which occur in a Stabilometer test.

Hveem and Davis (24) contend that bituminous pavements may be better classified in terms of their ability to resist deformation than by any other known means. They justify this by stating:

> ...Because roughness and corrugation of flexible pavements may develop before failure (in the sense of complete rupture) has occurred, there appears to be considerable justification for expressing resistance in the working range, regardless of forces required to produce ultimate disruption. This concept is being adopted by the California Division of Highways at the present time.

An outstanding feature of the Stabilometer method is that it has been in use for more than twenty-five years and much information is available to demonstrate the high degree of correlation between test results and known performance under motor vehicle traffic. Hveem and Vallerga (25) concluded that there is a high degree of correlation between Stabilometer results and pavement performance and little correlation between Stabilometer results and the density of the mixture except that the Stabilometer results are invariably low when the void spaces are filled or nearly filled with asphalt. This implies that the material under test is placed in a stress and strain condition simulating that which would occur in the road.

The test is conducted at a fixed rate of strain (0.05 inches per minute) to a maximum loading of 460 psi. The procedure calls for curing

of samples before compaction since the extent of curing will greatly influence test results when load is applied at a fixed strain rate. The lateral stress at a vertical load of 400 psi is recorded and used in the stability calculation. This load is assumed to reasonably represent stresses developed by truck traffic when impact is recognized in addition to static load. McCarty (28) reports a static load of 100 psi to be the maximum ever achieved under actual traffic and contends that the 400 psi reading will provide a safety factor of about four. Goetz, McLaughlin and Wood (16) present disagreement with McCarty since their tests indicate desirable confining pressures to be as high as 200 psi for Indiana-type mixtures. These results are based on thin specimens as the material is laid in the field. They generally concluded that the required confining pressure to produce a compressive strength equal to that developed by a thin layer of the mixture is equal to or greater than the unconfined compressive strength of the mixture.

The magnitude of the transmitted pressure in the Stabilometer is an inverse measure of the specimen stability. The air content has a critical effect on test results and an attempt is made to control this variable with a "displacement" measurement. Hveem adopted an initial displacement of two turns since test results were impaired considerably when lower values were used.

During the test the specimen becomes an integral part of the Stabilometer system and surface voids and air in the system will influence the lateral displacement required to develop a given pressure.

The entire Stabilometer procedure is empirical and this presents several drawbacks. It is dangerous to extrapolate test results to cover conditions beyond those established and it is difficult to avoid either

overdesign or underdesign. Overdesign or underdesign has been greatly reduced since the stability equation has been modified from a linear relationship between vertical pressure and transmitted pressure to a hyperbolic relationship. A wider range of acceptable stability values is possible when the following equation is used for calculating strength values rather than the linear relationship applied in early Stabilometer design work in California.

$$S = \frac{22.2}{\frac{P_h D}{P_v - P_h} + 0.222}$$

where S = Hveem stability

P_v = Vertical pressure = 400 psi

P_h = Lateral pressure corresponding to P = 400 psi (Stabilometer reading in psi)

D = Displacement value of pump required to increase lateral pressure from 5 psi to 100 psi while the specimen is being restrained from vertical movement.

The speed of testing, ease of operation, ease of adaptation to testing pavement cores and correlation information available provide an unmatched tool in the Stabilometer for strength testing of bituminous mixtures.

PURPOSE AND SCOPE OF THE INVESTIGATION

This study was undertaken with the intention of indicating a suitable Hveem design procedure to be employed in the design of Indiana bituminous mixtures under heavy traffic conditions. The study was two-fold in purpose:

1. To study the validity of using the present Hveem design method, as employed by the California Highway Division, for design of Indiana surface and binder mixtures as described in the State Highway Department of Indiana Specification Manual (45).

2. If some modification of the standard California procedure was indicated by step 1) as being desirable for Indiana surface and binder mixtures, the laboratory compaction pressure was altered until test results for field and laboratory specimens were in substantial agreement.

The study required sampling of bituminous pavements and performing a laboratory correlation. A mechanical kneading compactor was used for fabricating laboratory specimens to a density comparable to density results for cored specimens taken from the pavements. The Hveem Stabilometer was used for measuring the strength of both the field and laboratory specimens.

In evaluating the laboratory test results presented in this study it is important to view the data as an attempt to simulate the field condition of the pavement after five years of service by varying the kneading compaction intensity, and, in a few cases, the number of kneading tamps. No attempt is made to establish the most suitable asphalt

PURPOSE AND SCOPE OF THE INVESTIGATION

This study was undertaken with the intention of developing a [illegible] design procedure [illegible] to be employed in the design of bituminous mixtures under [illegible] conditions. The study was [illegible] in purpose:

1. [illegible] as employed by the California Division of Highways [illegible]

2. [illegible]

The study [illegible]

In evaluating the [illegible]

content by the Hveem design procedure since, in order to reproduce best the field properties of the mixtures, set asphalt contents and gradations are used which were established by tests on the field mixtures. The primary intention of this study was to simulate specific field conditions using a fixed mixture in the laboratory for each of four aggregate gradations.

MATERIALS

Specification limits for the aggregate gradations and mixture asphalt contents used in this study can be found in the specifications of the State Highway Department of Indiana for Hot Asphaltic Concrete (48). A description of the materials and presentation of test results for these materials are included in the following discussion.

Mineral Aggregate

In order to simulate the composition of the mixtures in service, construction record sheets on file in the Materials Laboratory of the Indiana Highway Department were reviewed in detail for each of the three contracts involved in this study. Table 2 records the source and size of the aggregate material for both binder and surface layers used for each contract. It will be noted that all of the coarse aggregate is crushed limestone, with the size variation from No. 11 in the surface to No. 9 or No. 8 in the binder.

Table 2 shows that several of the mixtures are identical with respect to size and source of aggregate and, therefore, it was decided to reduce the original six possible mixtures to four separate mix types for laboratory work, as shown in Table 3. This involved eliminating the McCook source of coarse aggregate and substituting Thornton Material, which seems justified since both the Thornton and McCook material are Niagara limestone from the Silurian geologic age. Thornton and McCook are located about twenty miles apart in the Valparaiso moraine country of northeast Illinois. Thus, the aggregate materials used in this study

Table 2

Sizes and Sources of Aggregates Used in Construction Projects

Highway	Layer	Material	Source
20	surface	No. 17 natural sand No. 11 crushed limestone No. 16 limestone filler	Elkhart, Indiana Thornton, Illinois Thornton, Illinois
20	binder	No. 17 crushed limestone sand No. 8 crushed limestone	Thornton, Illinois Thornton, Illinois
41	surface	No. 17 crushed limestone sand No. 11 crushed limestone	Thornton, Illinois Thornton, Illinois
41	binder	No. 17 crushed limestone sand No. 8 crushed limestone	Thornton, Illinois Thornton, Illinois
12	surface	No. 17 natural sand No. 11 crushed limestone No. 16 limestone filler	Elkhart, Indiana McCook, Illinois Thornton, Illinois
12	binder	No. 17 natural sand No. 9 crushed limestone	Elkhart, Indiana McCook, Illinois

Table 3

Aggregate Sizes and Sources Used in Laboratory Mixtures

Gradation	Simulates Gradation Used in Highways -	Layer	Material	Source
A	41	surface	No. 17 crushed limestone sand No. 11 crushed limestone	Thornton, Illinois Thornton, Illinois
B	20, 12	surface	No. 17 natural sand No. 11 crushed limestone No. 16 limestone filler	Elkhart, Indiana Thornton, Illinois Thornton, Illinois
C	20, 41	binder	No. 17 crushed limestone sand No. 8 crushed limestone	Thornton, Illinois Thornton, Illinois
D	12	binder	No. 17 natural sand No. 9 crushed limestone	Elkhart, Indiana Thornton, Illinois

were obtained from two sources: the natural sand was obtained from the Elkhart Sand and Gravel Company of Elkhart, Indiana, and the crushed limestone was obtained from the Materials Service Company of Thornton, Illinois. The mineral filler also was obtained from the Materials Service Company. Table 3, then, presents the materials and sources that were chosen for use in the laboratory phase of this study. Physical properties of the aggregate used for each mixture are recorded in Table 4.

The gradation of the aggregate for each mixture was the average of at least three gradations from asphalt-extracted samples taken at the plant during the time of construction. This information was made available through the Materials Laboratory of the Indiana Highway Department on daily report sheets. These samples were taken as the hauling trucks left the plant, usually as they passed over the weighing scales. The asphalt was extracted, and the aggregate from the sample was passed through a standard nest of sieves at the materials laboratory.

The average gradations obtained in this manner are presented in Tables 5 and 6 for the surface and binder courses, respectively. In Table 5 the laboratory gradations identified as Gradations A and B were chosen to correspond to the average gradations obtained for highways 41 and 12, respectively. In Table 6, corresponding binder gradations have been designated as Gradations C and D. Highway 20 gradations were not duplicated in the laboratory for two reasons: 1) the gradations for highway 20 check closely with one or the other of the highway 12 and 41 gradations, and 2) highways 20 and 41 represent similar non-channelized traffic flow conditions. A visual study of the traffic volume and type of vehicles on each highway indicated similar conditions between highways 12 and 41. Also, volume counts in 1957 reveal nearly the same num-

Table 4

Physical Properties of Aggregates (a)

Property	Gradation			
	A	B	C	D
Bulk Sp Gr (CA)	2.59	2.63	2.56	2.56
Apparent Sp Gr (CA)	2.72	2.71	2.70	2.70
Percent Absorption (CA)	1.86	2.01	2.05	2.05
Bulk Sp Gr (FA)	2.70	2.56	2.67	2.48
Apparent Sp Gr (FA)	2.77	2.60	2.78	2.59
Percent Absorption (FA)	0.98	0.97	1.46	1.12
Filler Sp Gr		2.72		
CKE, percent	3.4	1.9	4.0	1.9
OE, percent	3.5	3.5	3.9	3.2
Surface Area, sq ft/lb	28.5	17.9	24.7	10.4
Optimum Asphalt Content (percent by aggregate weight)	5.8(b)	5.5(b)	5.8(b)	5.8(b)

(a) Refer to Appendix E for complete test results.

(b) Extrapolation was necessary to determine these results from the nomographic charts used for optimum asphalt content in California.

Table 5

Sieve Analyses of Aggregate for Surface Mixtures

Sieve Size		Composition Limits (percent)		Highway 20 (percent)	Highway 41 (percent) Gradation A	Highway 12 (percent) Gradation B
Passing	Retained	Minimum	Maximum			
1/2 in.	3/8 in.	2	14	9.5	10.8	9.2
3/8 in.	No. 4	20	50	39.2	36.5	39.9
No. 4	No. 6	0	11	9.5	8.8	9.6
No. 6	No. 8	0	11	4.3	4.1	3.8
No. 8	No. 16	5	20	5.1	10.5	5.4
No. 16	No. 50	10	25	23.6	16.1	24.1
No. 50	No. 100	2	17	5.5	4.1	4.8
No. 100	No. 200	1	5	0.6	3.3	0.6
No. 200	---	3	5	2.7	5.8	2.6
Total Retained on No. 6 Sieve by Mix Weight		45	55	54.4	52.2	55.2
Number of Daily Record Gradations Averaged				6	3	6

Table 6

Sieve Analyses of Aggregate for Binder Mixtures

Sieve Size		Composition Limits (per cent)		Highway 20 (per cent)	Highway 41 (per cent) Gradation C	Highway 12 (per cent) Gradation D
Passing	Retained	Minimum	Maximum			
1 in.	1/2 in.	5	50	41.8	41.5	20.1 (a)
1/2 in.	No. 4	10	60	26.9	26.7	42.7
No. 4	No. 6	0	5	1.4	1.5	5.4
No. 6	No. 8	0	5	1.9	2.0	2.7
No. 8	No. 16	3	12	6.8	6.7	4.5
No. 16	No. 50	5	20	11.5	10.7	20.0
No. 50	No. 100	2	10	3.3	3.1	3.6
No. 100	No. 200	0	4	2.6	2.5	0.4
No. 200	---	0	4	3.8	5.3	0.6
Total Retained on No. 6 Sieve by Mix Weight		60	70	66.1	65.8	64.1
Number of Daily Record Gradations Averaged				6	8	11

(a) 3/4 in. maximum size.

ber of vehicles daily on these two highways.

Thus, Gradations A and C correspond to the surface and binder material in highway 41 and Gradations B and D correspond to the surface and binder material in highway 12. These gradings are illustrated graphically in Figures 1 and 2. It will be noticed that, to the nearest whole percent, these gradations are within specification limits for all sieve sizes with the exception of the minus No. 200 material for mixtures A and C. Tables 5 and 6 and Figure 1 show gradings A and C, which are surface and binder gradations, respectively, containing a higher percentage of minus No. 100 and No. 200 sieve material than Gradations B and D, surface and binder gradings, respectively, as shown in Tables 5 and 6 and Figure 2. Gradations A and C are composed entirely of crushed limestone and Gradations B and D are composed of crushed-limestone coarse aggregate with natural sand fine aggregate.

Bituminous Material

The asphalt used in preparing all laboratory specimens for this study was a 60-70 penetration grade asphalt cement furnished by the Texaco Company from their Port Neches, Texas refinery.

Table 7 records the physical properties of this asphalt and Table 8 lists the asphalt content for each mixture. Asphalt contents were determined by an average of three extraction results on samples from each pavement after five years of service. The three samples for each pavement were taken from each of three sections: one each at the intersection, 500 feet before the intersection, and 1000 feet before the intersection. All three samples were from the inside wheeltrack position.

It will be noticed, as in the case of the gradations chosen, that the

can be regarded daily as among the [illegible].

Thus, dimensions 1 and 2 correspond to the [illegible] and higher [illegible] [illegible] between 11 and dimensions 1 and 2 correspond to the surface and higher [illegible] to dimension 11. [illegible] [illegible] [illegible] in figures 1 and 2. [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] Tables 3 and 4 [illegible] [illegible] the surface and higher [illegible] with [illegible] [illegible] [illegible].

[illegible]

The [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible] [illegible]

Table [illegible]

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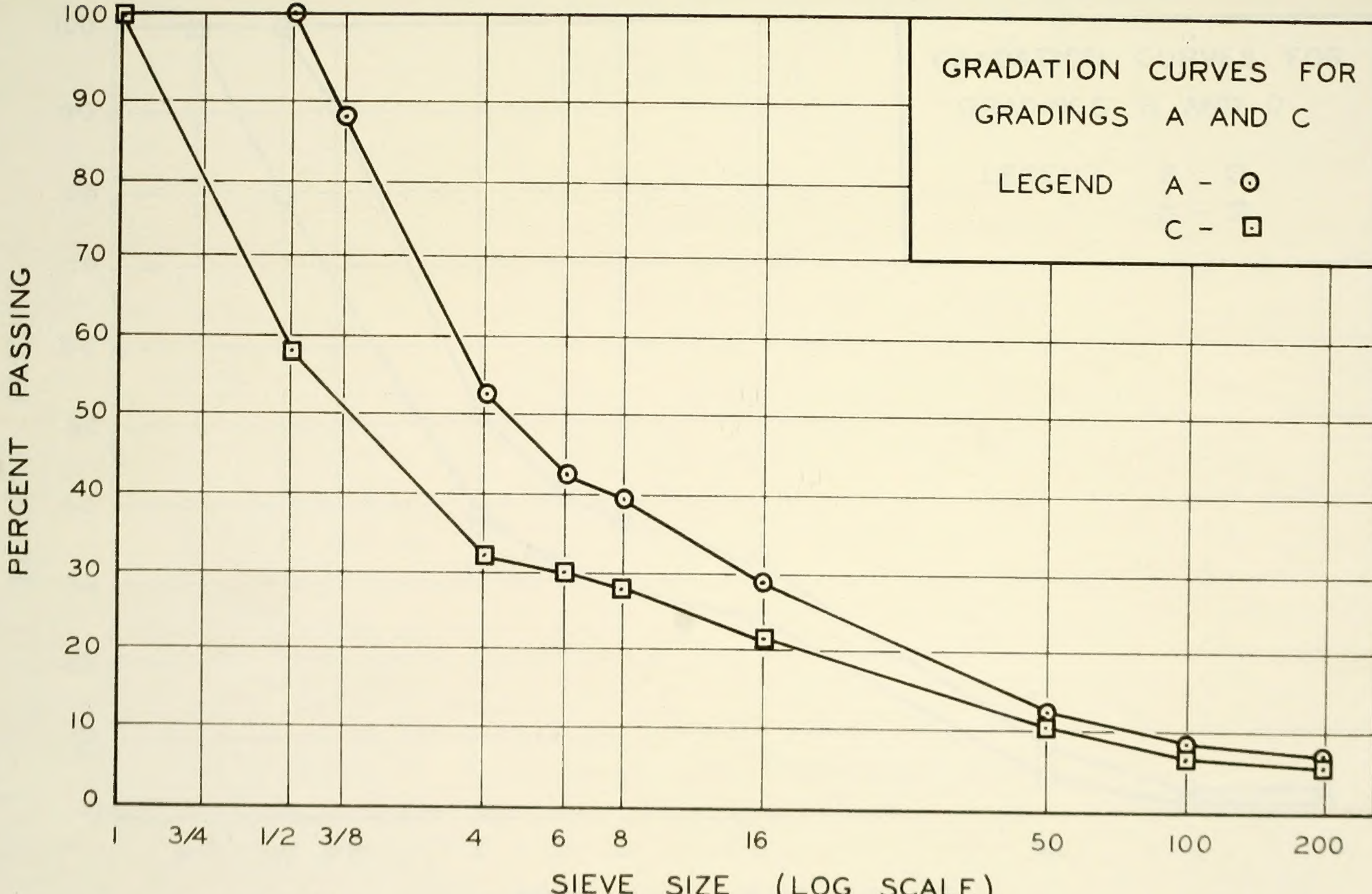

FIG. 1 AGGREGATE GRADATION CURVES

(CRUSHED LIMESTONE COARSE AND FINE AGGREGATE)

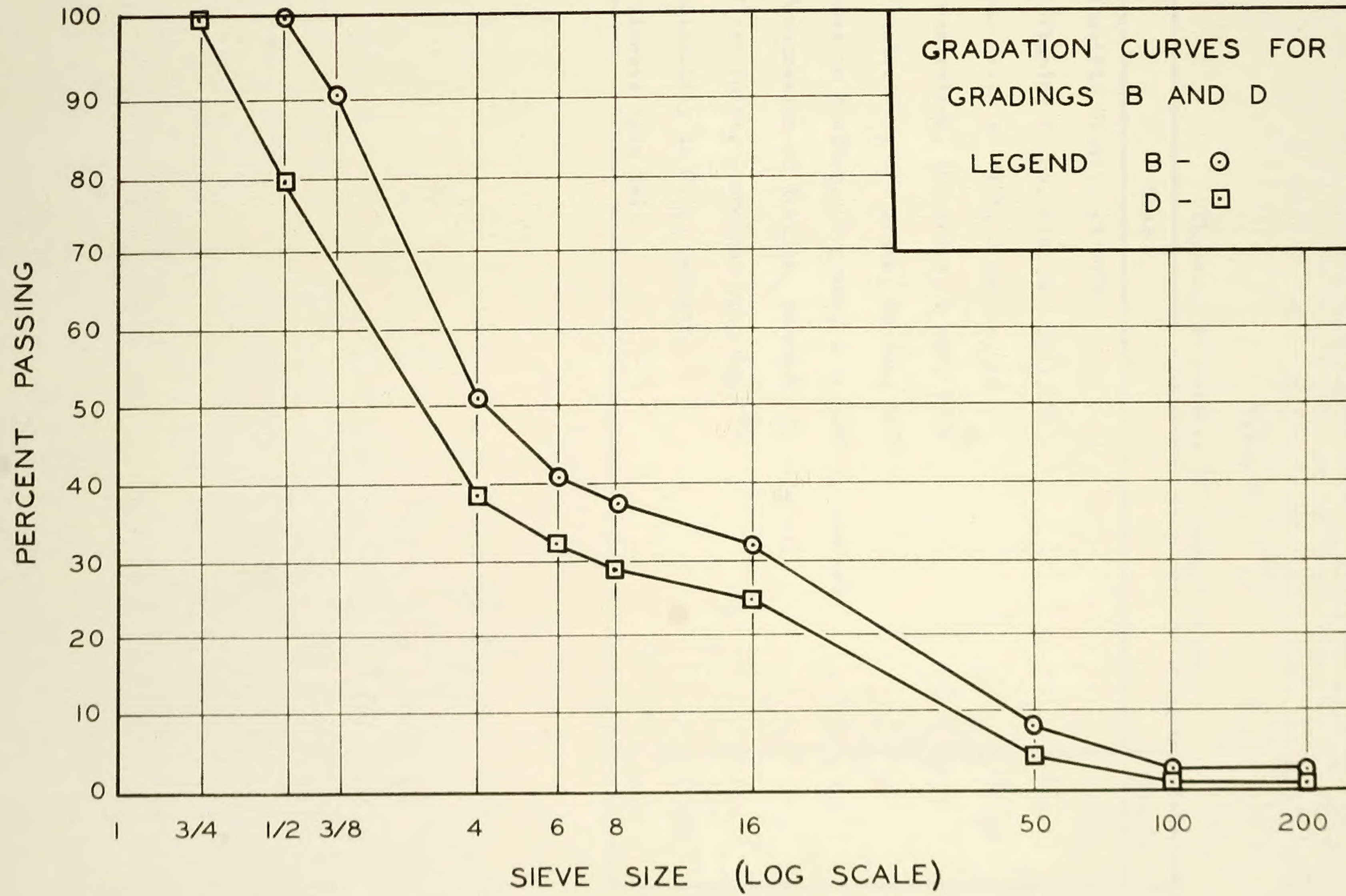

FIG. 2 AGGREGATE GRADATION CURVES
(CRUSHED LIMESTONE COARSE AGGREGATE, NATURAL SAND FINE AGGREGATE)

Table 7

Physical Properties of Asphalt Cement

Test	Result
Specific Gravity at 77°F	1.036
Softening Point, Ring and Ball, °F	124
Ductility at 77°F, 5 cm/min, cm	200 +
Penetration, 100 grams, 5 sec, 77°F	66
Penetration, 200 grams, 60 sec, 32°F	17
Loss on Heating, 50 grams, 5 hr, 325°F, percent	0.01
Penetration of Residue, percent of original	89
Flash Point, Cleveland Open Cup, °F	595
Solubility in CCL_4, percent	99.84
Oliensis Spot Test	Neg.

TABLE [illegible]

[illegible]

[illegible]	Test
[illegible]	Specific Gravity at 77°F
[illegible]	Softening Point, Ring and Ball, °F
[illegible]	Ductility at 77°F, 5 cm/min, cm
[illegible]	Penetration, 100 grams, 5 sec, 77°F
[illegible]	Penetration, 200 grams, 60 sec, 32°F
[illegible]	Loss on Heating, 50 grams, 5 hrs, 325°F, percent
[illegible]	Penetration of Residue, percent of original
[illegible]	Flash Point, Cleveland Open Cup
[illegible]	Solubility in CCl4, percent
[illegible]	Oliensis Spot Test

Table 8

Asphalt Contents for Various Gradations

Gradation	Highway	Section	Asphalt Content by Extraction (percent by mix weight)	Specification Limits, percent of mix wt	Chosen Asphalt Content, percent of mix wt
A	41	At intersection	6.8		
		500 ft before	7.4		
		1000 ft before	6.9		
			average = 7.0	5.5 - 7.5	7.0
B	20	At intersection	6.5		
		500 ft before	6.6		
		1000 ft before	6.5		
			average = 6.5	5.5 - 7.5	6.0
	12	At intersection	5.7		
		500 ft before	---		
		1000 ft before	6.2		
			average = 6.0		
C	20	At intersection	5.6		
		500 ft before	6.0		
		1000 ft before	5.6		
			average = 5.7	4.0 - 6.0	5.7
	41	At intersection	5.8		
		500 ft before	5.8		
		1000 ft before	5.6		
			average = 5.7		
D	12	At intersection	6.4		
		500 ft before	5.7		
		1000 ft before	6.2		
			average = 6.1	4.0 - 6.0	6.1

chosen asphalt contents for gradings A and C correspond to highway 41 and the asphalt contents for gradings B and D correspond to highway 12. Also, all asphalt contents are seen to be within specified limits, except that for Gradation D, which is only slightly above the maximum limit.

groups showed responses for questions 2 and 3 [illegible] Figures [illegible]
and the eighth graders [illegible] the greatest [illegible] in response to Figures [illegible].
Also, all eleventh graders were able to [illegible] Figures, ex-
cept that for [illegible] which [illegible]
[illegible].

PROCEDURES

The primary purpose of this section is to present the procedures used for handling and testing of both field and laboratory specimens. Detailed procedures for the various tests performed can be found readily in the literature. Reference to these procedures is made appropriately throughout the following discussion.

Field Procedures

Field operation procedures, and a description of the sampling locations are presented under this heading. The three highway resurfacings studied in this investigation were constructed in 1954 following Indiana specifications in use at that time. Four samplings of these pavements over a five year period have resulted in the data presented in Appendices D and E.

These pavement sections were selected for this investigation because they have been sampled periodically since their construction and because they represent a variety of heavy traffic conditions. These are Indiana AH resurface pavements consisting of an Indiana AH type B surface on an Indiana AH binder as described in the Indiana specifications (48). Details of these pavements follow.

Sampling Locations

The locations and boundaries of the three pavements studied are shown in Figure 3. The sections on U. S. 12 and U. S. 41 extend over a continuous length of 1.3 miles, for each highway, and the section

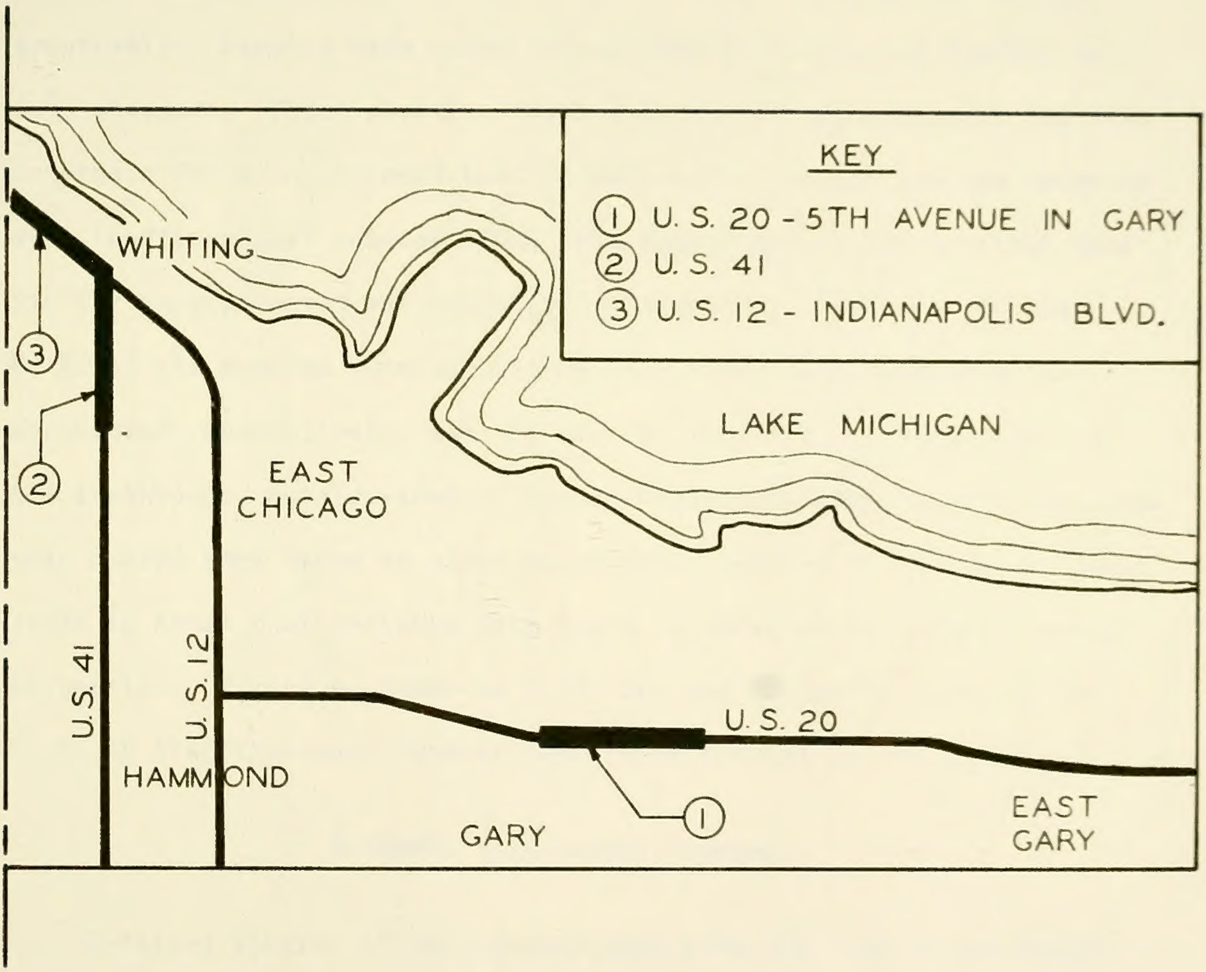

FIG. 3 CALUMET AREA - SAMPLING LOCATIONS

length on U. S. 20 is 2.0 miles.

Figure 38 and Table 23 of Appendix D present the general coring pattern and description of the location for each sample section, respectively. Samples were taken for similar conditions of traffic on each pavement. Three positions were selected at approximately 500 foot intervals for each pavement and, in each case, one section was selected at a traffic-signal intersection. The lane carrying the heaviest traffic for each pavement was selected for sampling. For U. S. 20 and U. S. 41 all samples were taken from the center lane, eastbound and northbound, respectively, and all samples for U. S. 12 were taken in the southbound, outside-traffic lane. Subsequent samples over the five-year period were taken as close to previous samples as reasonably possible in order that reliable data could be obtained for several years of service. Figure 4, taken on U. S. 20, and Figure 8, taken at the U. S. 41 traffic-signal intersection, show typical coring patterns.

Pavement Performance Studies

Detailed studies of performance were made over the entire length of each pavement location with particular emphasis on the pavement performance in sampling areas. For the most part, these studies were conducted immediately after the 1959 samples were obtained. Since the general coring locations had been pre-determined by earlier studies, it did not matter if performance studies were conducted before or after samples were taken. Photographs of the pavements shown in this section were taken during the summer of 1960, approximately six years after construction and one year after the last pavement sampling.

Table 9 lists general information about the pavements and traffic

Table 9

Pavement Data

Highway	Vehicles Per Day (1960) (a)	Indiana Aggregate Size No.	Layer	Design Thickness, in.
20	12,000 (both directions)	11, 17, 16	surface	1
		8, 17	binder	1-1/4
41	7,000 (northbound lanes)	11, 17	surface	1
		8, 17	binder	2 (two 1" layers)
12	16,000 (southbound lanes)	11, 17, 16	surface	1
		9, 17	binder	2 (two 1" layers)

(a) 1960 traffic data show approximately 20 per cent truck traffic for each of these highways.

volume to which each pavement is subjected. The traffic conditions on U. S. 20 and U. S. 41 are very similar: high-volume with a posted speed limit of 30 mph (12,000 vehicles daily for both directions on U. S. 20 and 7,000 vehicles daily in the northbound lanes on U. S. 41). U. S. 12 carries much higher traffic (16,000 vehicles daily in the southbound lanes) with a posted speed limit of 30 mph. The percentage of trucks on each highway is about 20 per cent or approximately 2,400 trucks in both directions for U. S. 20 per day, 1,400 in the northbound lanes for U. S. 41, and 3,200 in the southbound lanes for U. S. 12. Traffic on U. S. 20 and U. S. 41 is non-channelized in type as illustrated by Figures 4 and 5. The traffic on U. S. 12 is channelized into three lanes in each direction, as shown in Figure 6.

The traffic-signal intersections from which samples were taken for each pavement are shown in Figures 7, 8 and 9 for U. S. 20, U. S. 41 and U. S. 12, respectively. A general indication of the traffic volume and conditions of traffic are shown by these figures along with a general view of pavement performance at the three intersections. Figure 10 shows that no apparent rutting has occurred in the U. S. 20 pavement under the most severe traffic conditions in the sampled area. The rutting condition of the U. S. 41 pavement was found to be very similar to the U. S. 20 condition. The only severe rutting and shoving problem is on U. S. 12 at the intersection with U. S. 41. This condition is shown in Figure 11. Here rutting was found to be most serious in the outside-traffic lane and averaged about 3/4-in. after five years of service over a length of 250 feet approaching the intersection. Rutting, throughout the remaining length of the three pavements did not exceed about 1/4-in. in any case.

FIG. 4 NON-CHANNELIZED TRAFFIC CONDITIONS - U.S. 20

FIG. 5 NON-CHANNELIZED TRAFFIC CONDITIONS - U.S. 41

FIG. 6 CHANNELIZED TRAFFIC CONDITIONS - U.S. 12

FIG. 7 INTERSECTION - U.S. 20

FIG. 8 INTERSECTION - U.S. 41

FIG. 9 INTERSECTION - U.S. 12

FIG. 10 PAVEMENT CONDITION IN TRAFFIC LANE AT INTERSECTION - U.S. 20

FIG. 11 PAVEMENT CONDITION IN OUTSIDE LANE AT INTERSECTION - U.S. 12

Reflection cracks occur over all three pavements. They are most predominant on U. S. 12 and occur least frequently on U. S. 20.

Sampling Operations

The samples in 1954, 1955 and 1957 were obtained by using a 4-in. inside diameter drill and Carborundum abrasive was used to aid in the drilling process. The equipment used for obtaining cores in 1959 is pictured in Figure 12. A 4-in. inside diameter drill with a diamond bit was used for drilling. This bit cut a depth of about 1-in. for each minute of operation when cutting samples from the three pavements studied. Figure 13 shows the core height variation on U. S. 12. The variation at this location was more extreme than for the other two pavements studied.

For the first three samplings a total of eighteen samples were taken from each pavement for each sampling. Six samples were taken from each section, three in the wheeltrack position and three between wheeltracks. The number of samples was increased to fifty-four for the 1959 sampling since more laboratory tests were to be performed for the 1959 sampling than for the earlier samplings. Eighteen cores were taken at each of the three sections in the pattern shown in Figures 4 and 8. Of these eighteen samples, nine were taken in the inside-wheel-track position and nine were taken between wheeltracks.

The 1959 samples were obtained under hot and humid weather conditions and some difficulty was encountered with asphalt adhering to the drill. This was greatly counter-acted by the use of a soap solution in the water being used as a drilling aid.

The cores were immediately cleaned of adhering asphalt, identified,

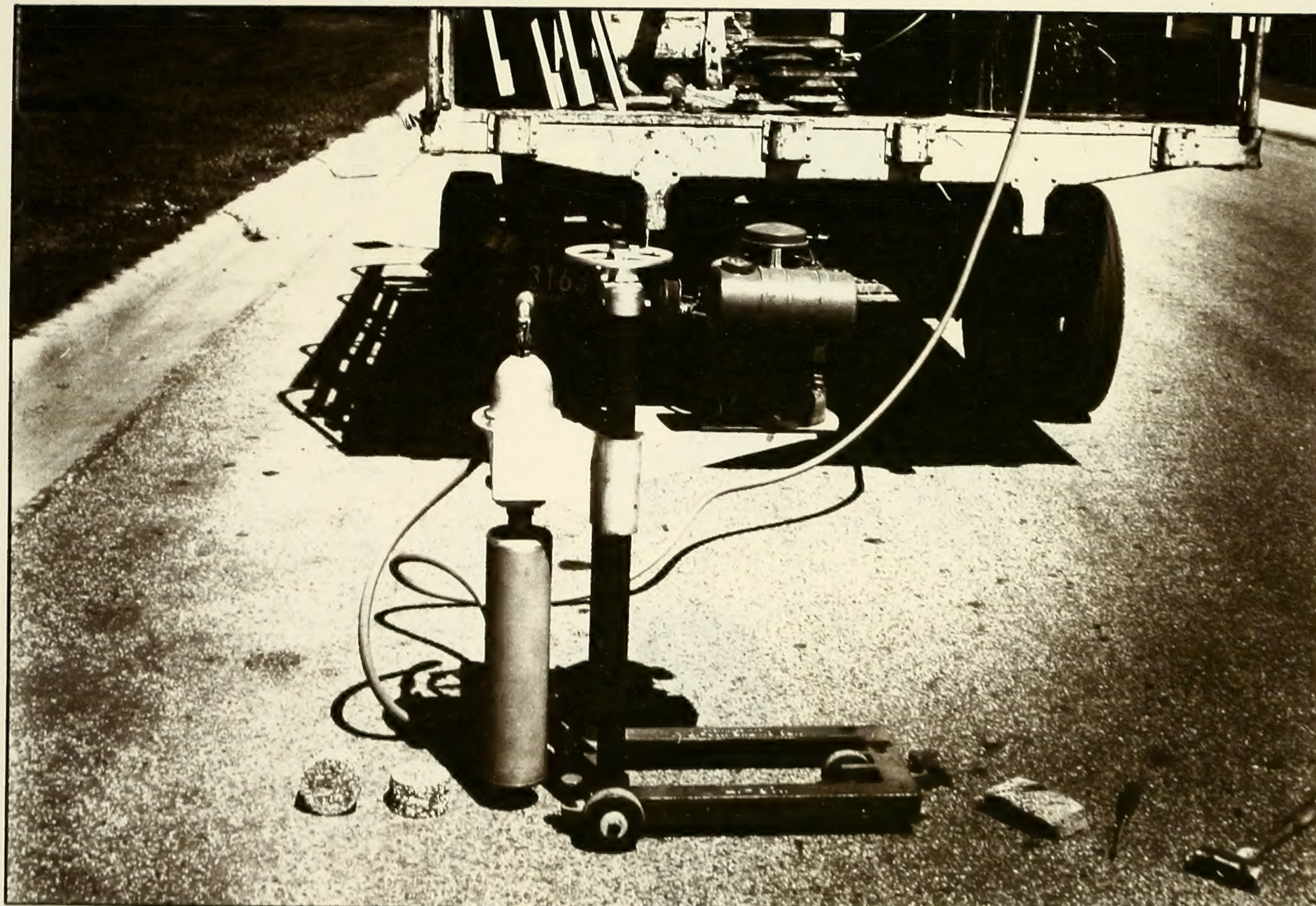

FIG. 12 CORING EQUIPMENT

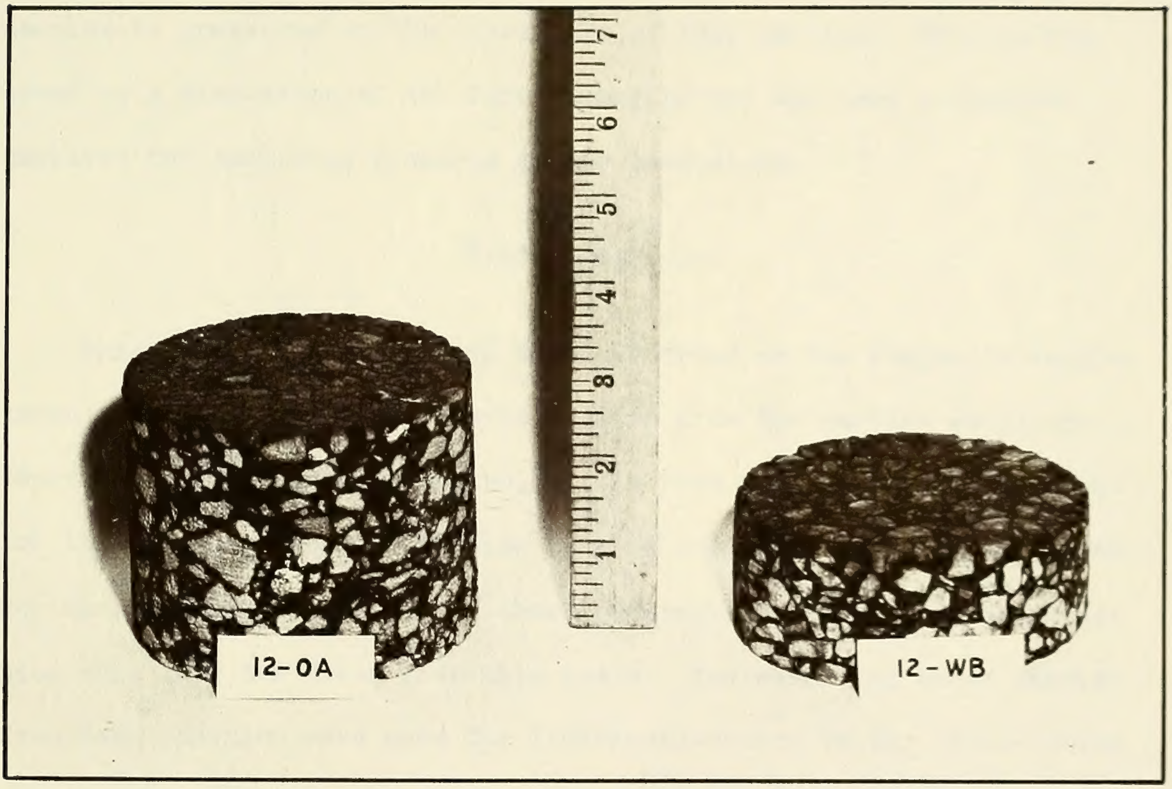

FIG. 13 CORE HEIGHT VARIATION - U.S. 12

and placed on a firm and level surface of the truck floor-bed for transportation to the laboratory.

Laboratory Procedures

The test procedures employed in the testing program for the field samples is presented in the first part of this section. This is followed by a discussion of the fabricating method and test procedures employed for specimens prepared in the laboratory.

Field Specimens

Density and Marshall tests were performed on the composite samples taken in 1959 for comparison with results from the earlier samplings. Twelve samples from each section, or six from each position, were used for these tests. Three composite samples from each position were used for Marshall tests and another three composite samples from each position were used for Hveem stability tests. The remaining three samples from each position were used for individual-course tests. These tests required separation of the surface and binder by use of a masonry saw. Figure 14 shows a special jig which was used to obtain proper horizontal and vertical alignment of the core to produce a cut parallel to the surface of the core.

Specimen Preparation. Before performing any tests it was necessary to allow the samples to reach and maintain a constant weight since they had absorbed water from the sampling operation. Three days was found to be sufficient time for this drying process at room temperature. During this time each sample was cleaned of foreign material and the composite height of each core was measured. This height was taken as an

and planes of a [illegible] and lower surface of the [illegible]

by the laboratory.

Laboratory Procedures

The test procedures employed in the testing program for the field samples is presented in the first part of this section. This is followed by a description of the [illegible] and test procedures [illegible] specimens prepared in the laboratory.

Field Samples

Cutting and Trimming. [illegible]

[illegible]

Figure 14 shows a [illegible]

and vertical alignment of [illegible]

surface of the core.

Specimen Preparation. [illegible]

[illegible]

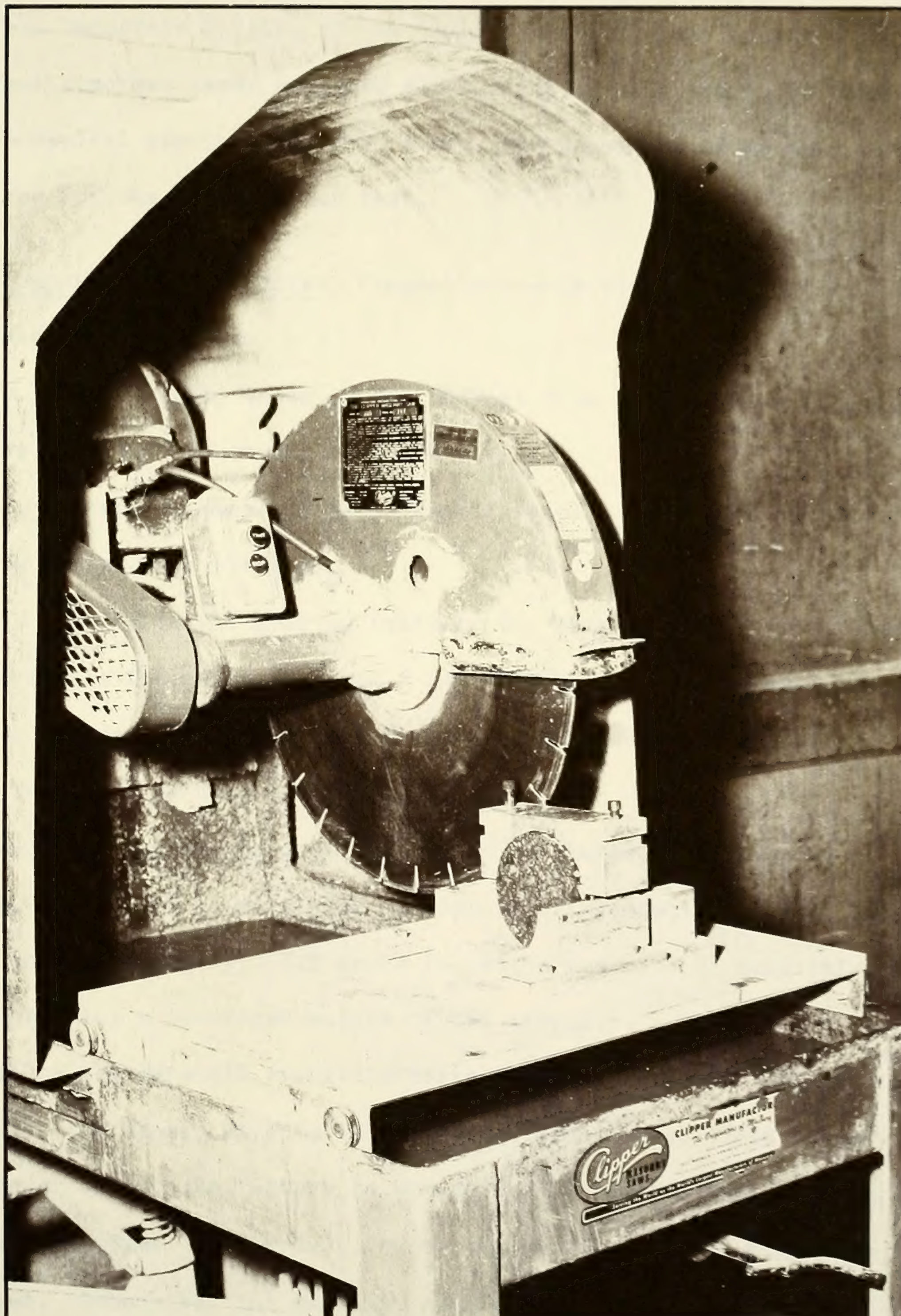

FIG. 14 JIG USED TO HOLD CORES FOR CUTTING

average of five readings using a special measuring device obtained from the Materials Section of the Indiana State Highway Department. The four specimen types prepared for testing in this study are shown by the typical specimens in Figure 15. The specimens shown have been prepared for the Stabilometer test.

Composite-Sample Tests. Composite-sample tests were performed in the following sequence:

1. Bulk density tests on six samples from each section or three samples from each position.

2. Marshall stability tests on all samples for which composite density values were obtained.

3. Rice specific gravity tests on two samples from each section or one sample from each position. (Previously used for Marshall tests).

4. Hveem stability tests on six samples from each section or three samples from each position.

Bulk density values were determined by the water displacement method, and in the usual manner, except that the samples were submerged in water approximately 1-1/2 hours to reduce the rate of absorption when obtaining a submerged weight of the sample.

Since the Marshall stability tests were performed on the specimen as-cored, the sample heights varied over a wide range. This necessitated the use of correction factors to obtain the corrected stability and flow for each specimen. For this purpose, the curves shown in Figure 16 were employed. Otherwise, the Marshall procedure followed the standard method as described by Marshall (27).

The Rice specific gravity test was used to determine the maximum

average of [illegible] specimens [illegible] representing [illegible] obtained from the [illegible] Division of the Corps of Engineers [illegible]. The four specimens [illegible] in this study are shown by the typical gradation [illegible] Figure [illegible]. The specimens [illegible] were prepared for the [illegible] tests.

Composite-Sample Tests. Composite-sample tests were performed in the following sequence:

1. Relative density tests on six samples, [illegible] from each position.

2. [illegible] tests on six samples [illegible] density values were determined.

3. [illegible] tests on the samples from each [illegible] position [illegible] (Table [illegible]).

4. [illegible] tests on [illegible] samples from each position.

[illegible] values were determined [illegible] method, [illegible] (approximately [illegible]) [illegible] obtained [illegible].

Since the [illegible] tests were performed [illegible], the sample [illegible]. This [illegible] from the [illegible]. For this purpose, the curves shown in Figure [illegible] were [illegible], as described by [illegible] ([illegible]).

The [illegible] test was used [illegible] the [illegible]

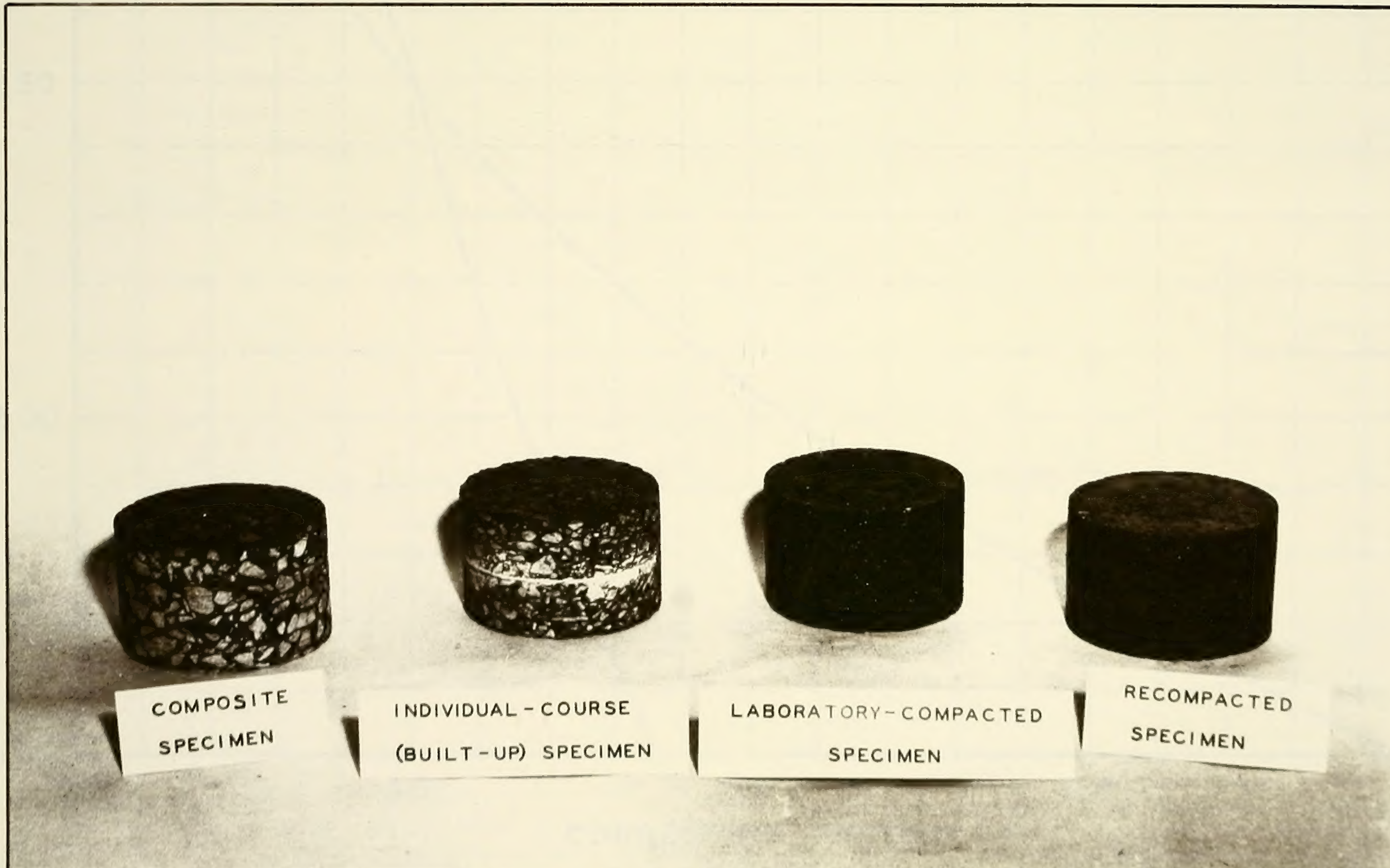

FIG. 15 TYPICAL TEST SPECIMENS

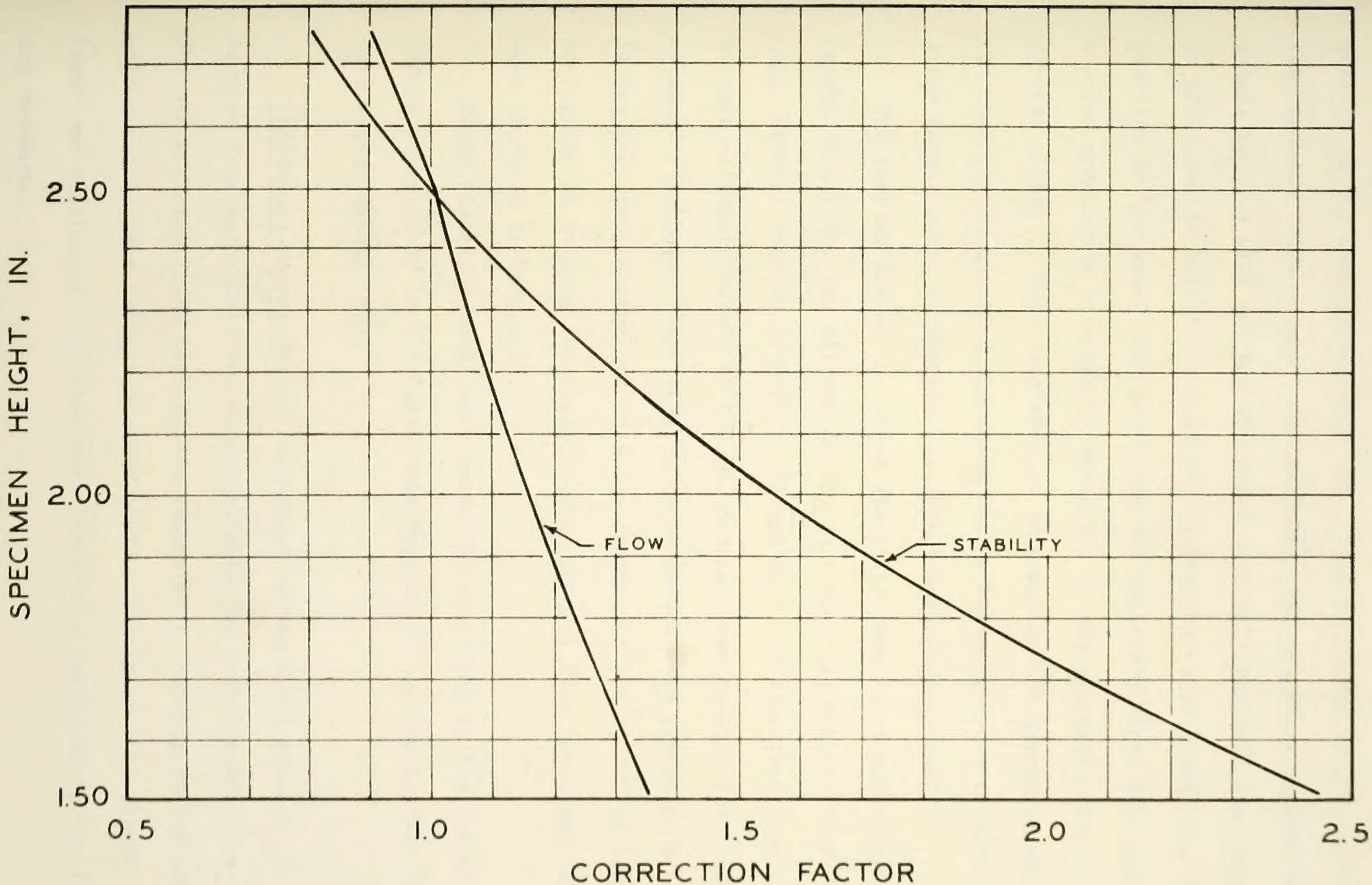

FIG. 16 MARSHALL CORRECTION CURVES
(FROM PURDUE LABORATORY DATA - SEPT. 15, 1955)

density of the composite-sample mix. This value was used to calculate the percent voids in the total mixture. The procedure is described in detail by Rice (39) and the procedure generally followed for this study is presented in Bulletin 105 of the Highway Research Board (5). An addition to the usual test procedure was necessary since the coring operation produced a specimen in which all of the aggregate was not coated with asphalt. This consisted of determining the amount of moisture absorbed by the uncoated aggregate during submergence by fan-drying the loose sample until it reached a constant-weight condition.

The samples tested for Hveem stability were first cut to a uniform height of 2.5 in. and allowed to dry to a constant weight before testing. Several samples from U. S. 12 and U. S. 20 were less than the minimum allowable height of 2.2 in. and no Hveem stability values were obtained for these. Each sample, to be tested, was placed in a circulating-air oven, controlled at a temperature of 140°F, for at least one hour prior to testing immediately after removal from the oven. This oven is shown in Figure 17.

Hveem stability tests were conducted according to the normal procedure as outlined in the California Manual (47) or the Asphalt Institute Design Manual (49).

Individual-Course Tests. One-third of the total samples, or three samples from each position, were available for individual-course tests. The surface and binder layers were separated by cutting at the interface with a masonry saw using a diamond blade. The in-service height of each layer was determined and laboratory tests were performed in the following sequence.

quality of the [illegible] were as reliable

the present [illegible]. The [illegible] of [illegible] in

detail by [illegible] followed the same

is presented in [illegible]. In

addition to the usual [illegible]

similar [illegible]

as with [illegible]. This [illegible]

absorbed by [illegible]

(see [illegible]

The [illegible]

method of [illegible]

192). [illegible]

minimum [illegible]

obtained the [illegible]

(1938-39) [illegible]

best [illegible]

[illegible] in Figure [illegible].

These [illegible]

method [illegible]

(see Design Manual [illegible]).

Irrigation-advance Tests. [illegible]

simpler [illegible], were available [illegible]

The [illegible] and [illegible] were [illegible]

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tests was [illegible] in the follow-

ing sections.

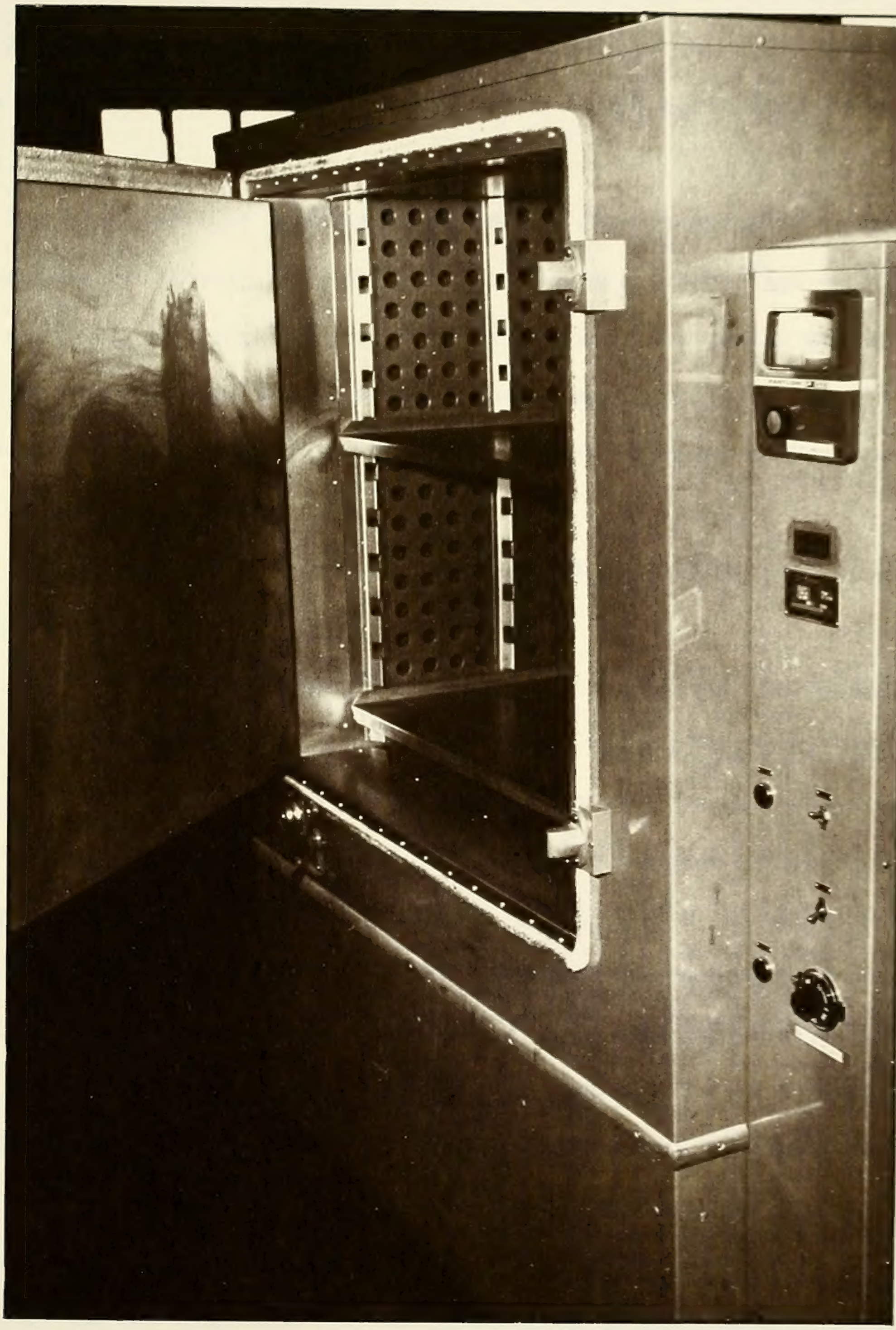

FIG. 17 CONTROLLED - TEMPERATURE OVEN - 140°F

1. Hveem stability tests on built-up specimens.

2. Bulk density tests on all layers used for the Hveem stability tests plus, for the binder material, those layers not tested for Hveem stability.

3. Rice specific gravity tests on the surface material taken from the samples previously tested for composite-Hveem stability.

4. Percent asphalt content by extraction, for binder and surface material from each of the three sections of each pavement, using the built-up specimens tested for Hveem stability. Plaster of paris was removed before testing.

5. Aggregate gradations for all samples from which asphalt was extracted.

The Hveem stability and Rice specific gravity tests were conducted using the same procedures as described above under Composite-Sample Tests. To obtain specimens of the required height for the stability test, it was necessary to combine three layers for a surface specimen and two layers for the binder material, using plaster of paris as a cementing agent between layers. The advantage of using plaster of paris is that, because of its light color, it can be distinguished easily from the asphalt-aggregate mixture if the cement is to be removed for further testing of the specimen. This method restricted the data to only one test result for each surface and binder layer in any position and, in several cases, especially for U. S. 12, an adequate amount of material was not available to form a specimen of the required height. After the stability test, the layers were separated and the plaster of paris was removed for a bulk density determination on each layer.

Six Rice specific gravity values were determined on surface ma-

terial for sections A and C of each of the three pavements. Representative samples were used from the wheeltrack positions only. The reason that only six Rice density values were determined was that an adequate amount of mix from any one position was not available to compose more samples. No Rice density values were determined for the binder layers.

The extraction procedure was that as outlined under AASHO designation T58 - 37: "Standard Method of Test for the Determination of the Percentage of Bitumen in Bituminous Mixtures", with dust correction modifications as discussed below.

Benzene was used as the solvent for extracting asphalt from surface samples from which asphalt would be recovered. Carbon tetrachloride was used to extract asphalt from the binder material.

If the asphalt was to be recovered for further testing, the dust correction was determined by centrifuging the entire extract solution for a two-minute period at 2,000 rpm. The dust collected as residue was taken as the total dust correction. The remaining dust corrections were reported as the weight of dust settling out of the extract solution in a twenty-four hour period.

The dried aggregate from each extraction was sieved through a series of standard sieves by mechanical shaking for a fifteen-minute period. The entire aggregate sample after extraction was used for each gradation sample. Each sieve was previously weighed empty and a final weight of sieve plus material was obtained after the sieving operation. The weight of aggregate retained on each sieve was determined and reported as the percentage retained between consecutive sieves. All dust removed by the extraction process was assumed to be material passing the

No. 200 sieve in the gradation analysis of each aggregate sample.

Tests on Recovered Asphalt. Asphalt was recovered from several surface samples using the procedure outlined by Abson (1). A total of nine asphalt samples were recovered and each was tested for consistency by penetration, softening point, and ductility using the appropriate ASTM procedure for each test. The samples tested were from sections A and C of each pavement with some duplicate tests performed on samples from the same section and position.

Laboratory Specimens

This discussion of laboratory specimens is applicable to all specimens formed using the four gradations described in the MATERIALS section. Test procedures employed follow the standard test methods except for the modifications presented in the following discussion.

Aggregate and Mixture Property Tests. Representative samples of each gradation or mixture were used to determine physical properties of the aggregate and mixture for each of the four gradations and mixtures tested. Specific gravities, both bulk and apparent, were determined in triplicate and reported with the twenty-four hour percent absorption. These tests were made for both fine and coarse aggregates used in all four gradations and for the mineral filler used in Gradation B. Standard ASTM test procedures were followed throughout.

Other tests included determining the surface area, CKE (Centrifuge Kerosene Equivalent) and OE (Oil Equivalent) values for each gradation in order to obtain an optimum asphalt content. The procedure followed was that as outlined in the California Manual (47) using the surface

area factors presented therein for determining aggregate surface areas of each gradation.

Average specific gravity values were used to determine theoretical maximum specific gravity values for each mixture. The effective theoretical maximum specific gravity was recorded as the average of the bulk and apparent values and to be compared with the Rice specific gravity. The Rice specific gravity was reported as an average of at least two tests and determined by the procedure outlined in Bulletin 105 of the Highway Research Board (5).

<u>Specimen Fabrication</u>. The aggregate for each gradation was washed and dried after sieving into the desired fractions. Samples were batched in a quantity that would produce a final specimen height of 2.4 in. to 2.6 in. after compaction. This was normally 1150 grams of aggregate, but the quantity varied slightly with variation in compaction pressure.

Mixing of the aggregate and asphalt was accomplished by heating the aggregate to 300^{o}F and the asphalt to 275^{o}F in a Peerless gas oven and mixing with a Hobart electric mixer (model N-50) for a two-minute period. These temperatures were chosen, rather than the temperatures normally specified for the Hveem method, for convenience and ease of handling. This mixing procedure was followed for all laboratory prepared samples.

The asphalt was added directly to the mixing bowl, containing the aggregate, on a one gram direct-reading Toledo scale of 10 kg capacity. The setup is pictured in Figure 18.

After mixing the mixture was placed evenly in an 11 x 7 x 1-1/2

FIG. 18 ONE GRAM DIRECT-READING SCALE

in. flat-bottom pan for curing in the controlled temperature oven at 140°F if the mixture was to be compacted using the kneading compactor. Samples prepared for compaction with the Marshall hammer were not cured. In the latter case, the mixture was placed directly from the mixing bowl into a pre-heated compaction mold. The material was compacted according to the normal procedure outlined by Marshall (27) and specimens were immediately cooled under running water for at least two minutes before being extruded from the mold.

Samples to be compacted with the kneading compactor were cured for at least fifteen hours, but normally about twenty hours, since the longer period was found to be more convenient. The samples were then placed in the gas oven and the mixture temperature was gradually increased to slightly above the required compaction temperature of 230°F. The material was placed in a pre-heated mold in two layers while the mold was held secure in the mold-holder. Each layer was rodded twenty times in the center and twenty times around the edge with a 1/2 in. diameter, bullet-nosed rod. The mold was then placed in position beneath the compaction foot and the proper compaction was applied. The compaction procedure, from this point, did not differ from that outlined in the California Manual (47) or Asphalt Institute Design Manual (49).

Routine Tests on Laboratory Fabricated Specimens. One specimen was compacted at each pressure with the standard spring in the kneading compactor and Hveem stability and bulk density values were obtained for each specimen.

Specimens compacted with the old spring in the kneading compactor were used for the following tests:

1. Marshall stability and flow, with bulk density values determined before the strength test, on one specimen at each compaction pressure.

2. Hveem stability, with bulk density values determined after the strength test, on three specimens at each compaction pressure.

3. Percent asphalt content by extraction and aggregate gradations for each Hveem specimen.

The following tests were conducted on Marshall-compacted specimens:

1. Marshall stability tests on three specimens with bulk density values determined before the strength test.

2. Hveem stability tests on three specimens with bulk density values determined after the strength test.

3. Percent asphalt content by extraction for the three Marshall-compacted and Marshall-tested specimens with gradations on the aggregate of each specimen.

The procedure for each of the above tests has been presented under the procedure for <u>Field Specimens</u>. Normally specimens were extruded from the mold directly into the Stabilometer test chamber for testing; however, in the later stages of testing, it seemed desirable to extrude the specimen into another mold of larger diameter and, then, place it immediately in the Stabilometer. This procedure eliminated any possibility of damaging the Stabilometer by excessive loading.

Rice specific gravity tests were made in triplicate for each mixture and the results were used in the voids analysis. This test was performed on samples immediately after mixing and before any manner of compaction had been applied to the mixture.

Specimen Uniformity. Several of the specimens compacted with the standard spring were further tested to determine the effects of kneading compaction on specimen uniformity. For this purpose, the four specimens compacted with the highest pressure for each gradation were used. Each specimen was first cut into three layers using a masonry saw. The layers were coated with paraffin after they had been allowed to dry to a constant weight at room temperature. Paraffin-coated bulk density values were determined for each layer in the manner outlined in the Asphalt Institute Design Manual (49).

Following the density determination one specimen in the maximum stability range was chosen from each of the four mixtures to determine asphalt content variation and aggregate breakdown characteristics throughout the specimen. The paraffin coating was removed from the three layers of each specimen and the asphalt was extracted following the procedure described under Individual-Course Tests. After thorough drying, the aggregate samples were sieved on a No. 6 sieve. The percent asphalt content by extraction and the percent of aggregate retained on the No. 6 sieve, on a total mix weight basis, were reported for top, middle, and bottom layers of each of the four specimens.

Remolded-Sample Tests. Many of the binder and surface specimens that had been built up and tested in the Stabilometer were recompacted with the kneading compactor and tested for density and Hveem stability. In several cases, the pavement cores tested for Hveem stability had been used for other tests and a sample from the same section and position was substituted.

The plaster of paris was removed from each sample to be remolded

after which the material was heated sufficiently to be broken into individual particles. The mixture temperature was then increased slightly above the required compaction temperature of 230°F to allow for cooling while placing the material in the mold. The sample was placed in the pre-heated mold in two layers and each layer was rodded twenty times in the center and twenty times around the edge with a one-half inch diameter, bullet-nosed rod. The compaction procedure, from this point, was as outlined by the California Manual (47) or the Asphalt Institute Design Manual (49). The mechanical compactor calibration, performed at the completion of this study, shows that a semi-compaction pressure of 275 psi was used with a peak compaction pressure of 595 psi for these remolded specimens.

Each specimen was placed in the controlled temperature oven at 140°F for at least one and one-half hour after compaction and before testing in the Stabilometer. These specimens were extruded directly into the Stabilometer test cell and tested in the normal manner (47, 49).

The Bulk density of each specimen was determined after allowing sufficient time for the specimen to maintain room temperature. The procedure was that as outlined above under <u>Composite-Sample Tests</u>.

RESULTS

In this section, the test results from this study are presented along with a discussion and evaluation. Summarized data are included with graphical illustrations of trends shown by the data.

The field-specimen test results are considered first in order to establish the requirements for design in the laboratory. Laboratory-specimen test results are then evaluated and a design procedure is established for each of four gradations. Comparisons of field and laboratory test specimens are presented to demonstrate effects of variation in the two types of compaction.

Complete and detailed test results are presented in Appendices D through G and, throughout this discussion, reference is made to tables and illustrations to be found in these Appendices.

Field Specimens

Three highway sections constructed in 1954 in the vicinity of Gary, Indiana were sampled for this investigation. The field study was conducted primarily to study the change in stability and density of bituminous concrete resurfacing over a period of several years while under severe traffic conditions, and to establish a design criteria based on these findings. For this purpose, studies have been conducted over a period of five years, during which time four samplings have been made. Properties of the resurfacing material were studied in relation to sample section, or distance from a traffic-signal intersection and in relation to the wheel-path of traffic, for each sampling period.

Composite-Sample Property Variation Over Five-Year Service Period

A summary of the density and Marshall data obtained on composite samples during the five-year performance study is presented in Table 10. Trends indicated by these data are illustrated in Figures 19 through 22.

Figure 19 shows property variation with service time if test results for the three sample sections are averaged. For all practical purposes, the density is near maximum after one year of service. It can be stated also that Marshall stability is near maximum after one year of service and remains nearly constant for the next four years. No relationship is apparent between Marshall flow and service time in any case. The comparison of test results for wheeltrack and between-wheeltrack samples in Figure 19 definitely illustrates a significant variation for the more channelized-traffic conditions on U. S. 12 as compared to conditions on U. S. 20 and U. S. 41 where traffic is also much lower in volume.

The maximum traffic-compacted density will vary with the type of mixture, but these data indicate that the density increased by approximately 6 pcf above the construction density. These data show the maximum Marshall stability, in general, was about double the stability of the mixture at the time of construction.

Figure 20 presents test results for wheeltrack samples only, showing the effects of service time and sample section on the mixture property variation. The uniformity illustrated by the U. S. 12 density curves justifies, to some extent, the manner of sampling employed, and shows the uniform pattern of compaction on U. S. 12 as compared to the zig-zag pattern obtained under non-channelized traffic of lower volume.

It is apparent that the highest density is at the intersection for

Table 10

Bituminous Concrete Properties Over Several Years of Service (a)

Specimen Identification (b)	Bulk Density, pcf				Marshall Stability, lbs				Marshall Flow, 1/100-in.			
	1954	1955	1957	1959	1954	1955	1957	1959	1954	1955	1957	1959
20-WA	142.2	152.2	152.5	153.4	750	2744	1959	2374	20.6	21.0	23.8	15.5
20-WB	142.7	148.7	149.9	150.8	738	1539	1922	1923	18.7	14.4	18.5	15.5
20-WC	147.7	148.0	151.1	150.0	1152	1507	1695	1522	23.3	15.1	27.4	16.3
20-OA	144.8	151.1	154.2	152.6	912	2071	2843	2255	19.0	18.3	19.4	17.8
20-OB	144.3	149.4	150.2	150.4	806	1757	1796	1941	21.7	15.8	18.0	19.2
20-OC	145.7	150.4	149.7	150.0	944	1793	1609	1639	21.0	14.7	27.0	11.8
41-WA	147.2	150.9	150.4	152.4	1603	2175	2507	2294	23.9	24.8	19.4	15.2
41-WB	142.5	148.3	148.7	148.2	841	1579	2155	1539	29.1	22.1	24.2	24.1
41-WC	142.3	147.3	145.4	146.8	972	1668	1685	1725	23.5	22.3	22.2	24.3
41-OA	145.3	151.0	152.9	151.2	1292	2165	2785	1919	26.3	22.4	15.7	21.5
41-OB	142.7	148.0	147.1	149.5	827	1681	1664	1946	22.7	19.8	25.7	23.6
41-OC	142.3	145.7	147.5	146.7	1058	1703	2141	1905	40.1	27.9	26.3	31.2
12-WA	145.2	151.5	153.2	153.5	703	1589	1855	1524	16.8	12.9	19.0	16.0
12-WB	147.5	150.3	152.4	152.4	704	1425	1513	1059	13.4	17.0	12.7	17.7
12-WC	148.1	149.4	151.1	151.4	817	1564	1363	1684	11.5	12.2	19.1	16.0
12-OA	148.3	151.4	152.7	152.6	1018	1496	1832	2008	15.6	15.9	14.9	12.2
12-OB	143.9	147.1	146.9	146.9	638	1105	994	562	12.1	12.1	14.9	12.3
12-OC	142.2	148.0	148.4	149.6	580	1263	921	920	15.8	11.1	15.4	11.4

(a) All values reported are for composite samples consisting of a binder and surface layer and are an average of three test results.

(b) W and O refer to sample positions (wheeltrack and between wheeltrack, respectively). A, B and C refer to sample sections (at intersection, 500 ft before, and 1000 ft before, respectively).

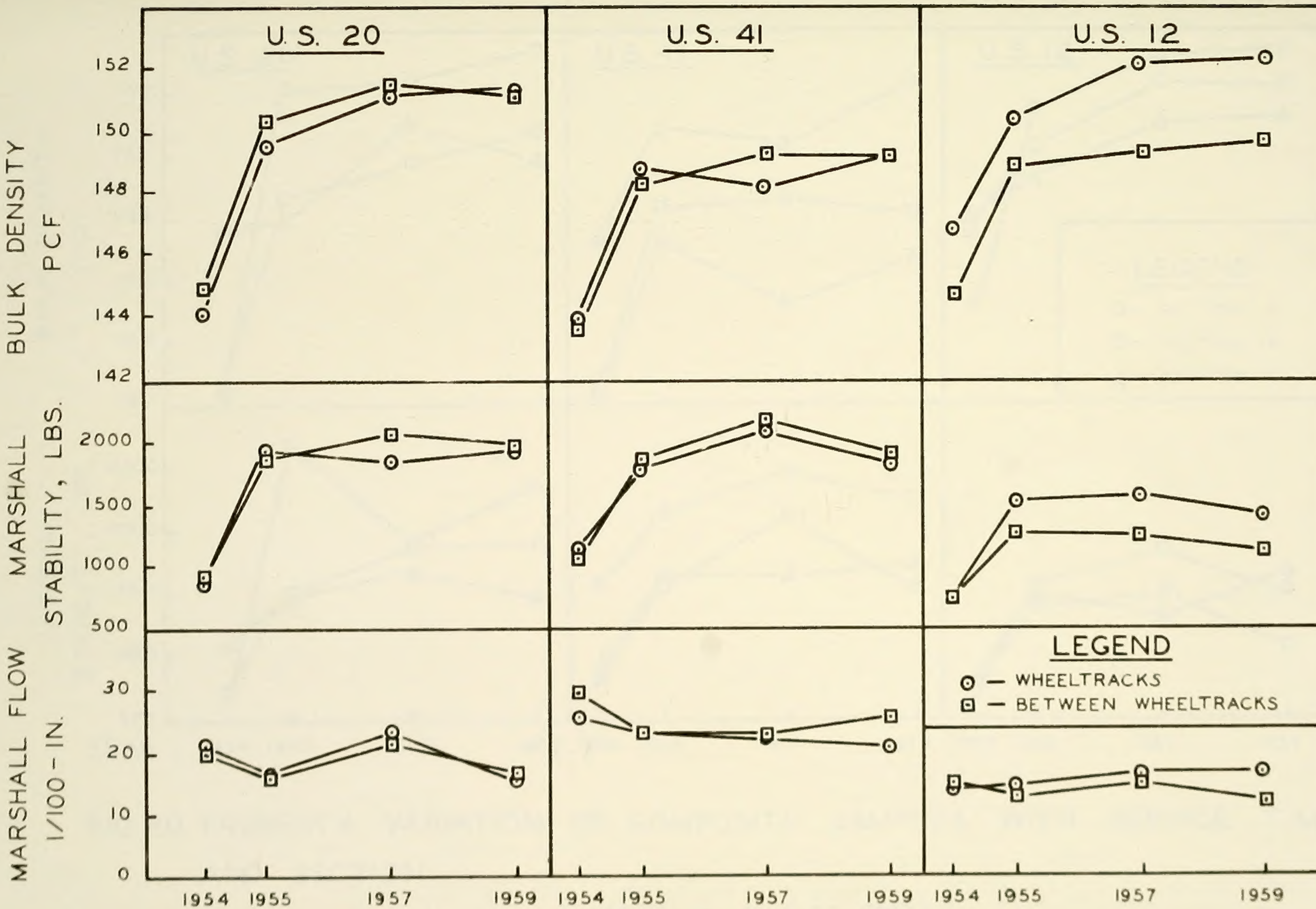

FIG. 19 PROPERTY VARIATION OF COMPOSITE SAMPLES WITH SERVICE TIME

(AVERAGE DATA - SECTION A, B, C SAMPLES)

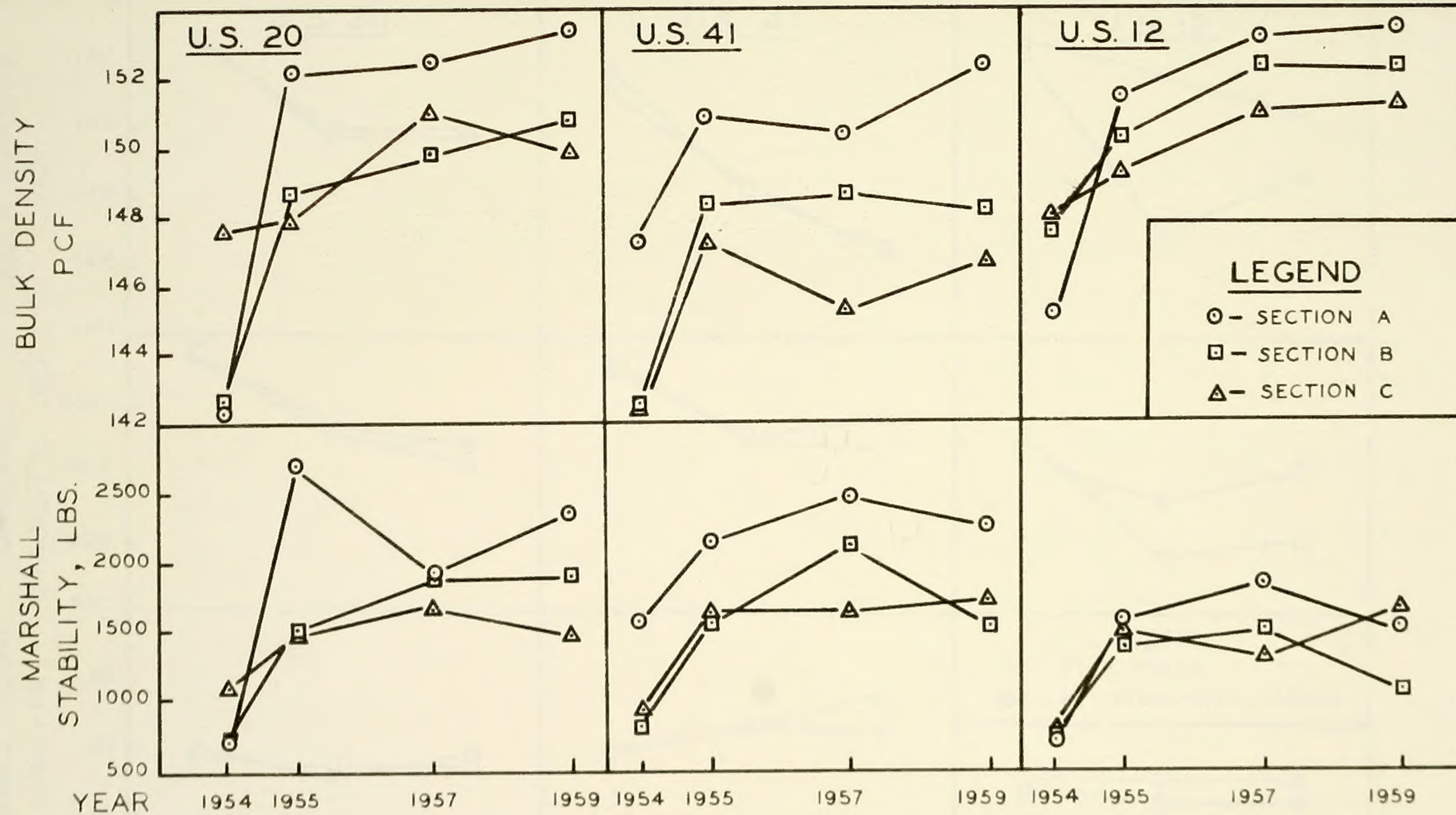

FIG. 20 PROPERTY VARIATION OF COMPOSITE SAMPLES WITH SERVICE TIME AND SECTION

(WHEELTRACK SAMPLES ONLY)

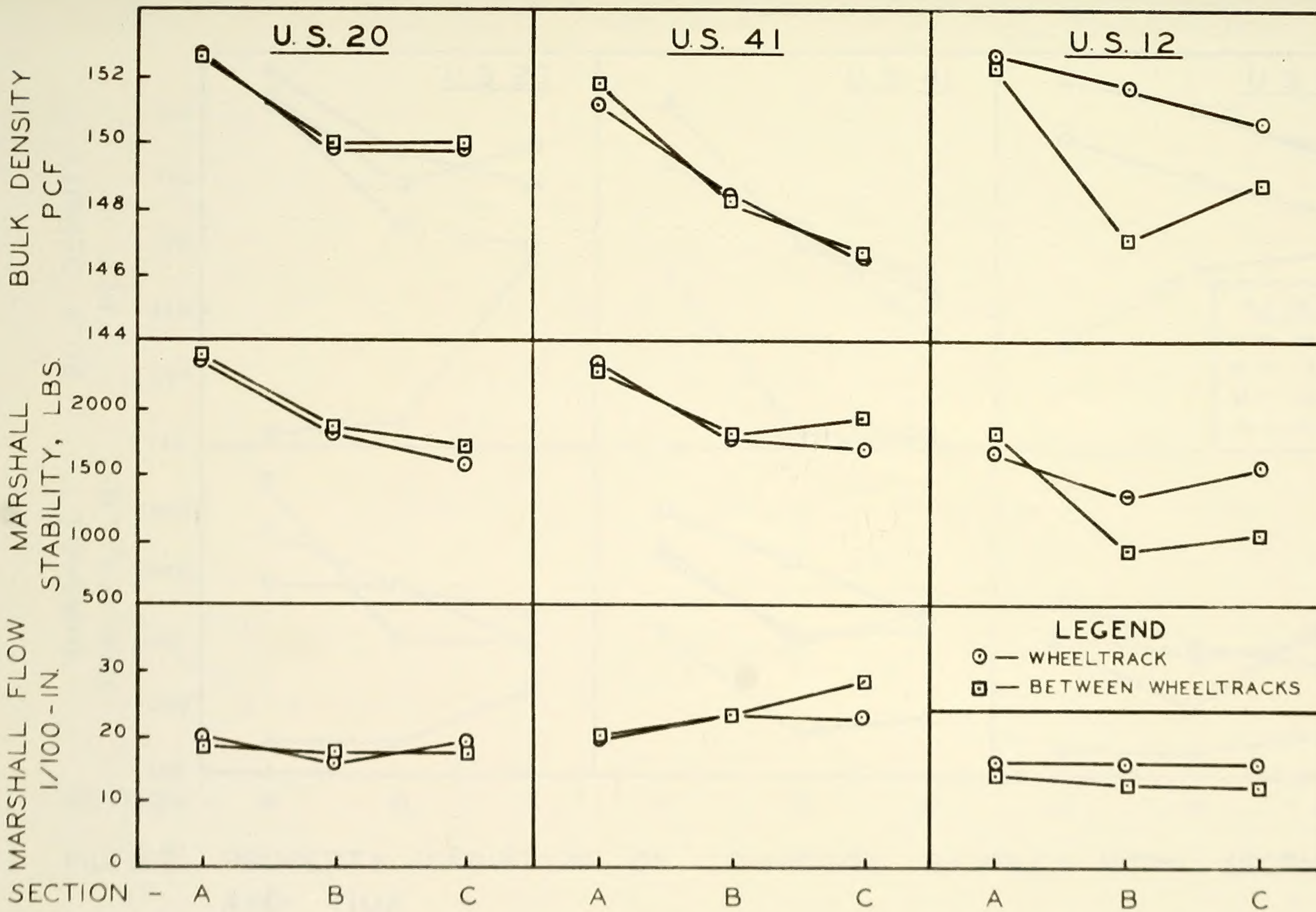

FIG. 21 PROPERTY VARIATION OF COMPOSITE SAMPLES WITH SECTION

(AVERAGE DATA - 1955, 1957, 1959 SAMPLES)

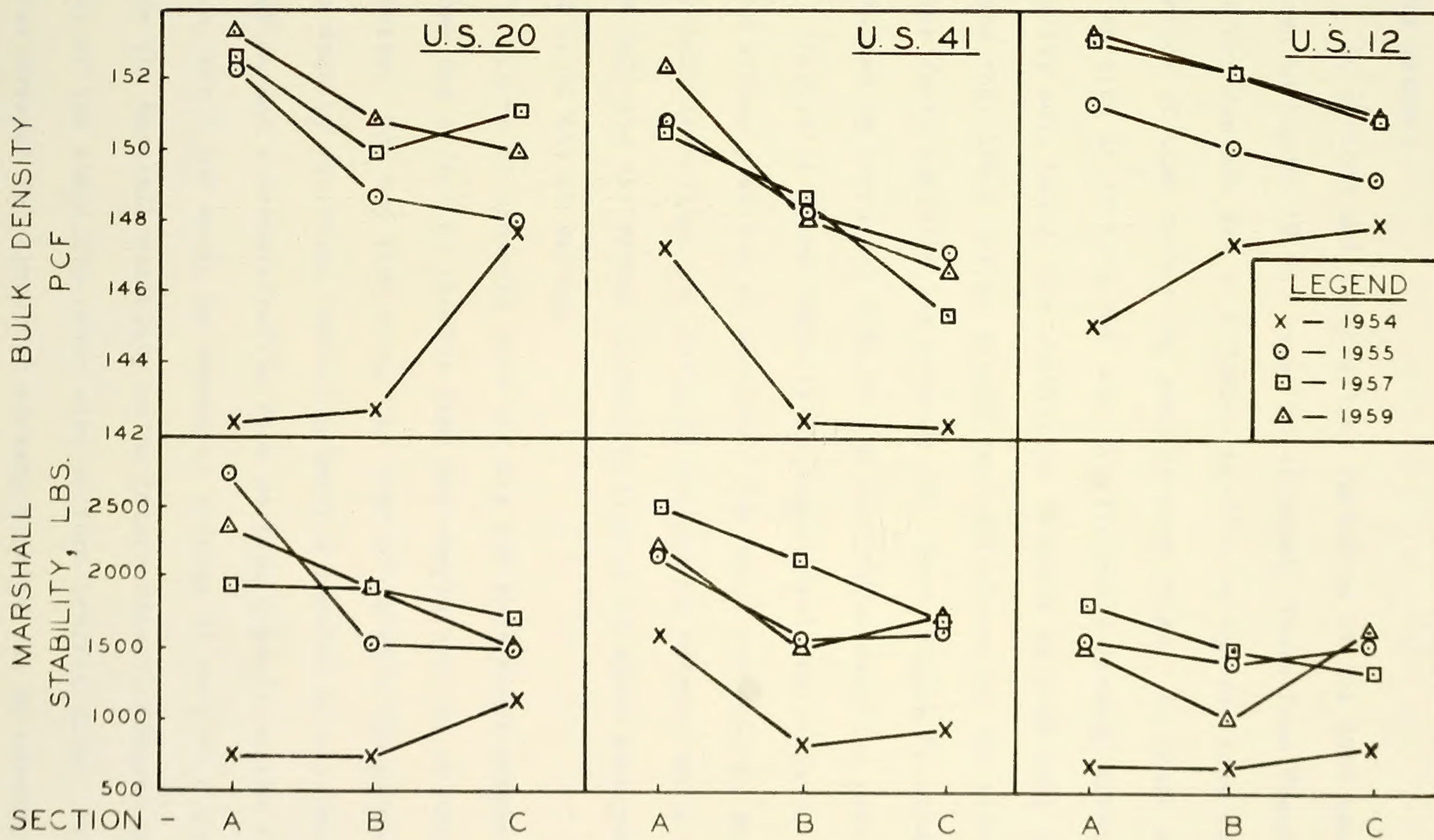

FIG. 22 PROPERTY VARIATION OF COMPOSITE SAMPLES WITH SECTION AND TIME

(WHEELTRACK SAMPLES ONLY)

each highway and that a gradual decrease in density results when moving further away from the intersection. Both Figures 20 and 21 illustrate this pattern.

An obvious and inconsistent variation of the 1954 test results shown in Figure 20 was cause to eliminate these from Figure 21. These differences are due to allowing traffic on the sections for varying periods of time before the samples were taken. In these early stages of service it is believed that traffic would greatly affect the mixture density and, hence, the stability. Studies by Zube (55) led him to state that there was no significant difference in the mixture density after forty vehicles has crossed it. Thus, Figure 21 shows patterns obtained by averaging data of the last three sampling periods.

Figure 21 shows there is no significant variation of flow value with either position or section. For other properties, density and Marshall stability, the definite difference between sections is evident. The definite difference between positions is again apparent on U. S. 20 and U. S. 41, as before.

Figure 22 presents test results for wheeltrack samples only and shows the effect of service time and sample section on the mixture properties. For any time after one year of service the graphs show that the density variation between a section located at a signalized-intersection and a normal-traffic flow section depends on the type of mixture, but 3 pcf would be common in a range of zero to 6 pcf. These data show the Marshall stability at an intersection increased up to 150 percent of the stability under more uniform traffic flow. Figure 22 pictures several relationships already pointed out by previous data showing the effects of service time, section, and position on the three

highway pavements studies. Perhaps the most significant fact shown by the data of Figure 22 is the randomness of results for the 1954 sampling which represents the as-constructed condition.

Composite-Sample Properties After Five Years of Service

For the 1959 sampling, after five years of service, a greater number of tests were performed on the samples than for previous samplings. Summarized results of these tests are tabulated in Table 11 and graphical illustrations of the test property variation with sample section are shown in Figure 23.

The Marshall test trends illustrated previously are again shown here. Values are highest for intersection locations. Hveem stability results do not readily show any correlation with the Marshall stability results and positive correlation with Marshall flow is not apparent. Also, the limited Hveem stability data do not allow a definite correlation to be established with pavement performance. However, all Hveem stability values are less than 35, which is specified as a minimum stability in California for laboratory compacted mixtures to be subjected to this type of high-volume traffic. A stability value of 35 seems to be unreasonably high for field-compacted samples from these pavements since adequate performance is indicated by far lower values. Table 11 shows a range of Hveem stability values from 12.8 to 28.2 for samples taken after five years of service from the three pavements. Only the U. S. 41 data are complete and the results show the stability at sections B and C to be nearly the same and the stability at section A to be about 150 percent of the value at either section B or C. For each section of U. S. 41 the stability is higher for between-wheeltrack sam-

[illegible] relations. Perhaps the most significant fact shown by the data of Figure [illegible] is the [illegible] for the [illegible] sampling which represents the [illegible].

[illegible]

The [illegible] sampling [illegible]

[illegible] of [illegible] the [illegible]

[illegible] results [illegible]

and [illegible]

are shown in Figure [illegible].

The [illegible]

[illegible]

results do not [illegible]

[illegible]

Also, the [illegible]

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[illegible] values [illegible]

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the [illegible] type [illegible]

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shows a [illegible]

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[illegible]

[illegible]

about [illegible]

[illegible]

Table 11

Summary of Composite-Sample Test Results
(1959 - After Five Years of Service)

Specimen Identification	Composite Height, in.	Marshall Stability, lbs	Marshall Flow, 1/100-in.	Hveem Stability	Bulk Density, pcf	Percent Mix Voids (a)
20-WA	2.33	2374	15.5	24.0	153.4	2.0
20-WB	2.26	1923	15.5	21.8	150.8	2.5
20-WC	2.69	1522	16.3	25.3	150.0	5.2
20-OA	2.08	2255	17.8	----	152.6	2.1
20-OB	2.15	1941	19.2	----	150.4	2.6
20-OC	2.52	1639	11.8	24.8	150.0	3.4
41-WA	2.81	2294	15.2	18.6	152.4	2.5
41-WB	3.28	1539	24.1	12.8	148.2	1.7
41-WC	3.01	1725	24.3	13.3	146.8	3.6
41-OA	3.14	1919	21.5	22.4	151.2	2.8
41-OB	2.90	1946	23.6	14.5	149.5	2.9
41-OC	3.49	1905	31.2	15.1	146.7	4.7
12-WA	2.69	1524	16.0	13.1	153.5	1.4
12-WB	1.53	1059	17.7	----	152.4	0.3
12-WC	2.20	1684	16.0	----	151.4	2.1
12-OA	3.10	2008	12.2	26.9	152.6	2.3
12-OB	2.04	562	12.3	----	146.9	5.1
12-OC	2.55	920	11.4	28.2	149.6	3.2

(a) Determined by use of Rice maximum density.

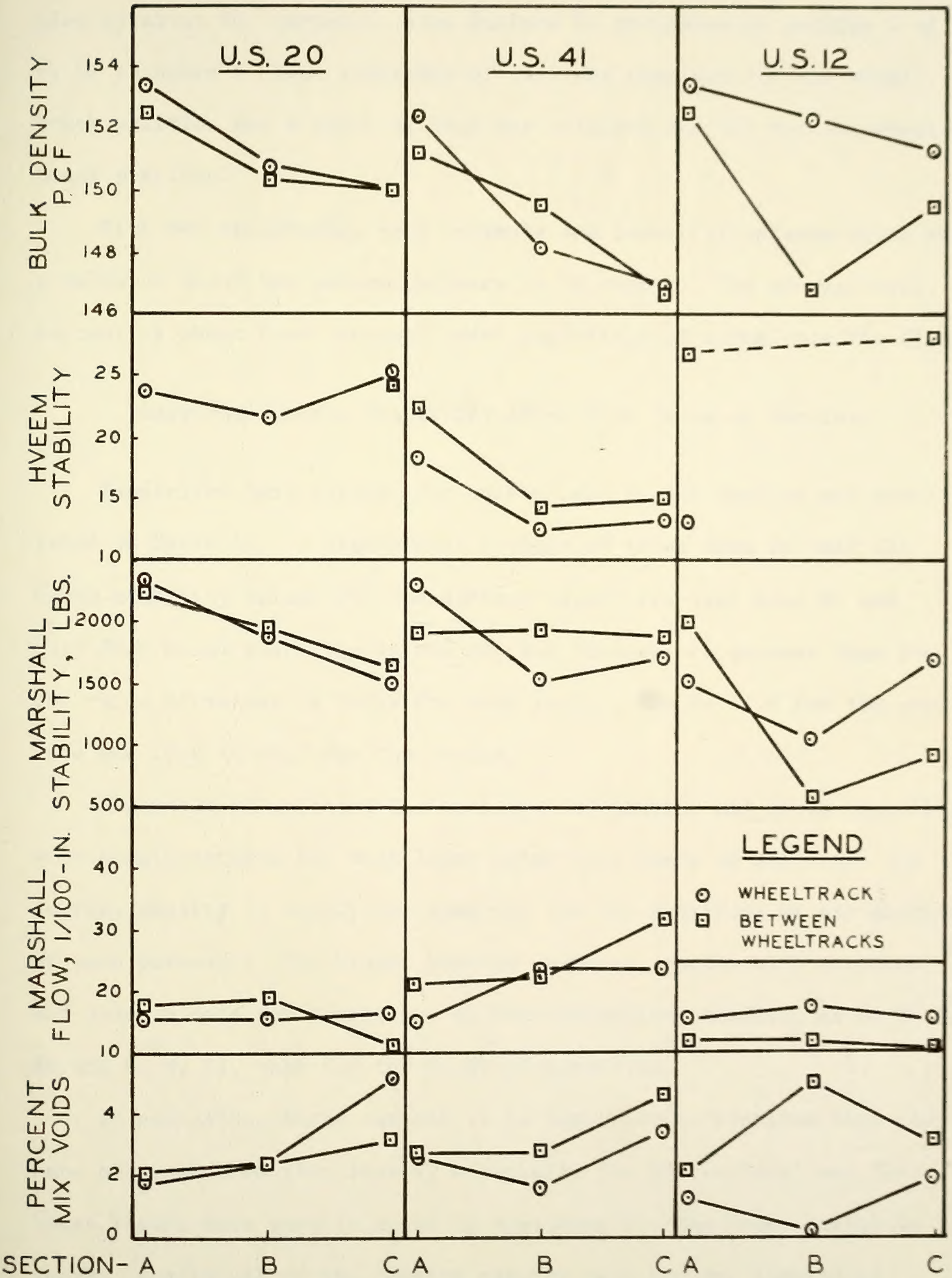

FIG. 23 VARIATION OF COMPOSITE - SAMPLE PROPERTIES WITH SECTION AFTER FIVE YEARS OF SERVICE

ples by about 20 percent. Some failure is indicated at section A of U. S. 12 where a Hveem stability of 13.1 was obtained for the wheel-track position and a value of 26.9 was obtained for the between-wheel-track position.

With two exceptions, void contents are lowest at intersections and a value of about two percent appears to be common. The average void content is about four percent under conditions of normal traffic flow.

Individual-Course Properties After Five Years of Service

Summarized test results for surface and binder samples are tabulated in Table 12. A significant feature of these data is that all Hveem stability values for the surface layers are less than 35 and only four Hveem stability values for the binders are greater than 35. The range of values is large for both layers, 5.4 to 34.8 for the surface and 15.2 to 48.7 for the binder.

Figure 24 illustrates the variation of density and Hveem stability with sample section for each layer after five years of service. The surface density is nearly the same for the two positions at any section of each pavement. The binder density, however, varies with position and does so more for conditions of non-channelized traffic, as on U. S. 20 and U. S. 41, than for the U. S. 12 condition.

In evaluating these results it is important to consider that they were obtained from thin layers, especially for the surface, and that these layers were used to build up specimens for the Hveem stability tests. Nearly all of the density results reported for individual courses were obtained after the Stabilometer test was performed. A statistical comparison using these data indicates that the density of

plus by about 20 percent. [illegible] is indicated by [illegible] to 8. It shows a [illegible] that was obtained for the [illegible] trench position and a value of [illegible] was obtained for the [illegible] trench position.

With two exceptions, [illegible] a value of about [illegible] contours is about [illegible]

Individual [illegible]

Numerical [illegible] listed in Table [illegible]. A significant [illegible] trench stability [illegible] only four trench [illegible] The range of values [illegible] zero and [illegible]

Figure [illegible] with gentle [illegible] surface density [illegible] of each [illegible] and down [illegible] 20 and [illegible]

In evaluating these results [illegible] were obtained from such figures, [illegible] these layers [illegible] trench. Nearly all of [illegible] values were obtained [illegible] statistical comparisons [illegible]

Table 12

Summary of Individual-Course Test Results
(1959 - After Five Years of Service)

Specimen Identification	Height, in.		Hveem Stability		Bulk Density, pcf		Percent Mix Voids	Percent Asphalt Content in Mix by Extraction		Percent Aggregate in Mix Retained on No. 6 After Extraction	
	Surface	Binder	Surface	Binder	Surface	Binder	Surface	Surface	Binder	Surface	Binder
20-WA	0.87	1.50	15.9	29.3	153.4	153.9	---	6.5	5.6	52.7	60.2
20-WB	0.84	1.40	19.1	24.7	150.0	152.5	---	6.6	6.0	52.5	58.8
20-WC	0.89	1.78	25.3	28.0	151.2	153.1	---	6.5	5.6	49.2	60.2
20-OA	1.01	1.04	34.8	29.0	153.2	150.2	0.7	---	---	---	---
20-OB	1.01	1.13	23.2	22.0	151.5	147.2	---	---	---	---	---
20-OC	0.77	1.76	28.9	23.4	150.8	---	1.4	---	---	---	---
41-WA	0.62	2.12	5.4(a)	25.9	153.5	153.1	---	7.3	5.8	48.9	62.3
41-WB	0.72	2.50	---	15.2	152.2	149.2	---	7.4	5.8	46.9	63.3
41-WC	0.77	2.31	10.0	22.4	151.8	149.0	---	6.9	5.6	48.3	66.4
41-OA	0.45	2.86	---	36.6	153.9	150.6	1.5	---	---	---	---
41-OB	0.71	2.11	---	18.5	152.3	147.6	---	---	---	---	---
41-OC	0.82	2.71	---	21.6	151.1	146.9	0.9	---	---	---	---
12-WA	0.72	1.84	---	24.0	153.4	153.0	---	5.5	6.4	52.8	59.2
12-WB	0.39	1.14	---	40.6	151.9	150.6	---	5.6	5.7	53.6	61.7
12-WC	0.79	1.38	---	33.6	152.1	151.8	---	6.2	6.2	52.6	57.1
12-OA	0.86	2.23	34.0	48.7	152.3	151.8	2.0	---	---	---	---
12-OB	0.33	1.71	---	27.8	151.6	150.9	---	---	---	---	---
12-OC	0.77	1.80	---	35.4	151.4	148.4	1.3	---	---	---	---

(a) Results were extrapolated when the lateral pressure exceeded the maximum dial reading of 200 psi.

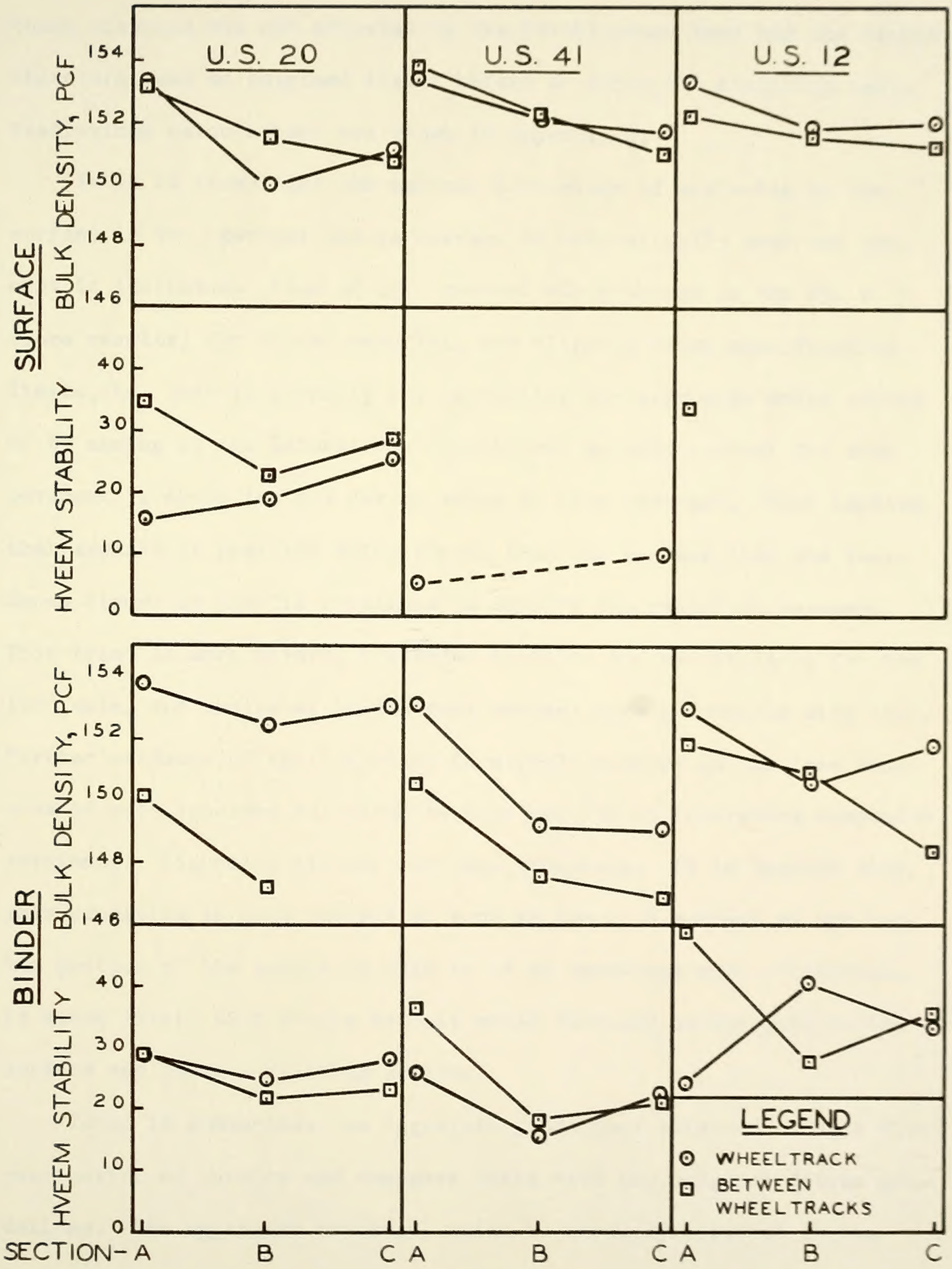

FIG. 24 HVEEM PROPERTY VARIATIONS WITH SECTION FOR INDIVIDUAL LAYERS AFTER FIVE YEARS OF SERVICE

these mixtures was not affected by the Stabilometer test and the density, therefore, can be obtained either before or after the stability test. Statistical calculations are shown in Appendix H.

Table 12 shows that the maximum percentage of mix voids in the surface is two percent and an average of only slightly over one percent is indicated. Some of the percent mix retained on the No. 6 sieve results, for binder material, are slightly below specification limits, but this is probably due to cutting the aggregate while coring or by sawing in the laboratory. The binder asphalt content for each pavement is above the mix design value of five percent. This implies that asphalt is possibly being forced from the surface into the less-dense binder as traffic continues to densify the resurface pavement. This trend is more clearly presented in Table 13, particularly for the 1959 data, and indicates this effect becomes more pronounced with time. Further evidence of this increase in asphalt content in the less dense area of each specimen was given by test results on laboratory compacted specimens. Figure 29 illustrates this phenomena. It is thought that this condition is only present as long as the void content of the bottom portion of the sample is high as in an open-type mix. Otherwise, it seems likely that excess asphalt would find an easier path to the surface and cause a flushing action.

Table 14 summarizes the aggregate gradations obtained after a five-year period of service and compares these with the original design gradations. The aggregate breakage, which is strongly apparent in the maximum size for the binder, is probably due to initial compaction. Another significant feature of these data is the obvious increase in the minus No. 200 sieve material for the binder and surface layers of

Table 13

Summary of Mix Asphalt Content and Percent Mix Retained on No. 6 Sieve (a)

Highway	Layer	Percent Asphalt Content in Mix				Percent Mix Retained on No. 6 Sieve			
		Design	1954(b)	1957(c)	1959(d)	Design	1954(b)	1957(c)	1959(d)
20	surface	6.8	6.7	6.2	6.5	53	54	---	51
20	binder	5.0	5.2	5.0	5.7	65	66	---	60
41	surface	7.0	7.1	6.8	7.2	53	53	---	48
41	binder	5.0	5.3	5.7	5.7	65	66	---	64
12	surface	7.0	6.9	5.9	5.8	53	54	---	53
12	binder	5.0	5.1	5.2	6.1	65	64	---	59

(a) Asphalt content limits for design are $\pm$ 0.3 percent. Percent retained on No. 6 sieve limits for design are $\pm$ 3 percent.

(b) 1954 samples were taken at the mix plant immediately after completion of the mixing process. Each test value is an average of three to eleven test results taken from daily report sheets for tests performed at the plant by Indiana Highway Department personnel.

(c) 1957 samples were from between-wheeltrack positions. Each value is an average of three test results with samples from all three sections for each pavement.

(d) 1959 samples were from wheeltrack positions. These samples were taken as near to the 1957 sample area as conditions of the pavement would permit. Each value is an average of three to five test results with samples from all three sections for each pavement.

Table 14

Aggregate Gradation Variation From Original After Five Years of Service (a)

Sieve Size		Percent Between					
Passing	Retained	U. S. 20		U. S. 41		U. S. 12	
		Original	After Five Years	Original	After Five Years	Original	After Five Years
Surface							
1/2 in.	3/8 in.	9.5	7.6	10.8	7.0	9.2	6.6
3/8 in.	No. 4	39.2	37.2	36.5	34.9	39.9	39.5
No. 4	No. 6	9.5	10.0	8.8	10.1	9.6	10.0
No. 6	No. 8	4.3	5.5	4.1	6.1	3.8	5.3
No. 8	No. 16	5.1	8.4	10.5	11.0	5.4	9.0
No. 16	No. 50	23.6	20.2	16.1	16.9	24.1	19.4
No. 50	No. 100	5.5	5.3	4.1	4.9	4.8	5.4
No. 100	No. 200	0.6	1.3	3.3	2.5	0.6	0.9
No. 200	---	2.7	4.5	5.8	6.6	2.6	3.9
Binder							
1 in.	1/2 in.	41.8	18.5	41.5	29.0	20.1 (b)	8.6 (b)
1/2 in.	No. 4	26.9	43.0	26.7	35.5	42.7	47.9
No. 4	No. 6	1.4	1.9	1.5	3.4	5.4	6.7
No. 6	No. 8	1.9	2.7	2.0	2.5	2.7	4.1
No. 8	No. 16	6.8	8.5	6.7	7.5	4.5	5.8
No. 16	No. 50	11.5	12.3	10.7	10.7	20.0	19.8
No. 50	No. 100	3.3	3.6	3.1	3.6	3.6	4.4
No. 100	No. 200	2.6	3.1	2.5	2.2	0.4	0.7
No. 200	---	3.8	6.4	5.3	5.6	0.6	2.0

(a) Five-year samples were taken in the wheeltrack position. Data are averaged for sections A, B, C and each gradation is an average of three to five test results. Original gradations are averaged from three to eleven gradations taken from daily record sheets prepared at the time of construction. Original samples were taken at the plant and before rolling.

(b) 3/4 in. maximum size.

each pavement. This is probably due to initial compaction and traffic, and possibly, to mixing.

A discussion and evaluation of some of the aggregate degradation phenomena in pavements and under kneading compaction is presented in another section.

Service Requirements

The complete field results of this investigation have been reduced to the final summaries appearing in Tables 15 and 16. They are labeled as "service requirements" because they are viewed as the test values required for the service that resulted in each case. The values in these tables were obtained by averaging only wheeltrack-sample test results for the three sections sampled since the type of service imposed on the wheeltrack is thought to be a more severe condition from the standpoint of stability requirements than the service condition to which the areas between wheeltracks are subjected.

The most desirable approach to establish a laboratory design procedure based on service requirements would be to consider the varying effects of traffic at sections A, B and C. However, the data obtained by this investigation are not thought to be extensive enough for each section to assign required test values for each traffic condition involved. Considering the Stabilometer test speed as representing the field condition where traffic is moving slowly over the pavement, this condition is best represented at some point between the end of normal-traffic flow movement and a point where traffic has stopped. It is thought that this condition may best be represented at section B, if any one section is to be chosen. However, the traffic pattern and

Table 15

Composite-Sample Service Requirements
(Average Data - Wheeltrack Samples Only; Sections A, B, C)

After One Year of Service.

Highway	Bulk Density, pcf	Marshall Stability, lbs	Marshall Flow, 1/100-in.
20	149.6	1930	16.8
41	148.8	1807	23.1
12	150.4	1526	14.0

After Three Years of Service.

Highway	Bulk Density, pcf	Marshall Stability, lbs	Marshall Flow, 1/100-in.
20	151.2	1858	23.2
41	148.2	2115	21.9
12	152.2	1577	16.9

After Five Years of Service.

Highway	Bulk Density, pcf	Marshall Stability, lbs	Marshall Flow, 1/100-in.	Hveem Stability	Percent Mix Voids
20	151.4	1939	15.8	23.7	3.2
41	149.1	1852	21.2	14.9	2.6
12	152.4	1422	16.6	13.1 (a)	1.3

(a) Section A only.

Table 16

Individual-Course Service Requirements - 1959
(Average Data - Wheeltrack Samples Only; Sections A, B, C)

Highway	Layer	Corresponding Laboratory Gradation	Bulk Density, pcf	Hveem Stability	Percent Mix Voids (a)	Percent Mix Retained on No. 6 Sieve
20	surface	B	151.5	20.1	0.7	51.5
20	binder	C	153.2	27.3	---	59.7
41	surface	A	152.5	7.7	1.2	48.0
41	binder	C	150.4	21.2	---	64.0
12	surface	B	152.5	----	1.7	53.0
12	binder	D	151.8	32.7	---	59.3

(a) Samples taken in the between-wheeltrack position. The other test results reported here are for wheeltrack samples.

volume vary for each of the three pavements and speed, stopping position, and acceleration and deceleration patterns are unequal between the pavements. Considering these factors and the data available, it is believed that an average of the wheeltrack data for the three sections, with equal weight given to each section, provides as good service requirement values as can be obtained from these data and for the traffic conditions represented by the three pavements. This procedure gives the test values presented in Tables 15 and 16. However, for the most severe conditions, such as that represented by the wheeltrack position in the outside lane on section A of U. S. 12, a somewhat more stable mixture is required and the field data are inadequate to establish a proper level.

Table 16 shows that the density of the binder on U. S. 20 is higher than the surface density, which is probably the result of using one-inch maximum size aggregate for the 1-1/4 in. binder layer. The low Hveem stability value of 7.7 for the surface on U. S. 41 appears to be completely satisfactory from the standpoint of pavement performance over the five-year period, but it is believed that this probably would not be true under the more severe conditions of channelized traffic as on U. S. 12 and it is possible that stability difficulties may yet develop with time.

Recovered Asphalt

Table 32 of Appendix E presents the results of several tests for consistency of the asphalt extracted and recovered from surface samples taken in sections A and C of each pavement. These data show a penetration range of 34 to 52 with an average value of 44 for nine test samples.

The average softening point was 131°F with a range of 127 to 137 and the average ductility for the nine asphalt samples was 112+ cm for a range of values from 90+ to 150+.

These results indicate that no unusual problems exist with the asphalt material. Duplicate tests for samples from the same section and position show significantly different penetration results such as 34 and 45 for samples from section 41-WC. Sufficient data are not available to conclude that the asphalt properties varied excessively.

Laboratory Specimens

The following discussion includes a presentation of test results for all specimens molded in the laboratory using both the standard and old springs in the kneading compactor and using the Marshall compaction method. The results obtained with the standard spring are representative of a smaller number of test specimens than those for the old spring, but they are preferred since with the standard spring the mechanical compactor was operating in the proper manner and applying the intended type of compaction. Therefore, these results are shown graphically in this section and the results using the old spring have been grouped in Appendix G as a supplementary section. Some varying features of the two types of compaction are discussed and comparisons are made between these and, also, with the Marshall method results.

In evaluating the laboratory test results in this study it is important to view the data as an attempt to simulate the field condition of the pavement after five years of service by varying the kneading compaction intensity, and, in a few cases, the number of kneading tamps. No attempt is made to establish the most suitable asphalt content by the

Hveem design procedure since, in order to reproduce best the field properties of the mixtures, set asphalt contents and gradations are used which were established by tests on the field mixtures. However, data are presented in Tables 33 through 36 of Appendix E which which show an optimum asphalt content of 5.8 percent by weight of aggregate, or 5.5 percent by weight of mix, for Gradations A, C and D and 5.5 percent by aggregate weight, or 5.2 percent by mix weight, for Gradation B using the Centrifuge Kerosene Equivalent and Oil Equivalent test procedures. Asphalt contents actually used for the work done in this investigation were 7.0, 5.7 and 6.1 percent by weight of mix for Gradations A, C and D, respectively, and 6.0 percent for Gradation B. It would be expected that a Hveem design series would indicate an optimum asphalt content of near 5.5 percent by weight of mix for Gradations A, C and D and near 5.2 percent by weight of mix for Gradation B, but this was not determined in the laboratory since the primary intention of this study was to simulate specific field conditions using a fixed mixture for each of four aggregate gradations.

Routine Tests

The routine tests include those for density, stability, and mixture voids using the kneading compactor with both springs and using Marshall compaction, and aggregate degradation tests for kneading compaction with the old spring and for Marshall compaction. Four mixtures were used in this laboratory phase of the study and they have been identified by the aggregate gradation they contain. Gradations A and B are surface gradations of 1/2 inch maximum size and passing the requirements for an Indiana type B bituminous concrete mix. Gradation A is

composed entirely of crushed limestone aggregate with 7.0 percent asphalt and Gradation B is composed of crushed limestone coarse aggregate and a natural sand fine aggregate with 6.0 percent asphalt. Gradations C and D pass specifications for Indiana binder mixes. Gradation C has a maximum particle size of one inch in a gradation composed entirely of crushed limestone aggregate with 5.7 percent asphalt. Gradation D has a maximum particle size of 3/4 inch and is composed of crushed limestone coarse aggregate and a natural sand fine aggregate with 6.1 percent asphalt. All asphalt contents are of 60-70 penetration grade asphalt cement and by weight of mixture.

Standard Spring. Complete test results using the standard spring are presented in Table 37 of Appendix E. Figures 25, 26 and 27 illustrate these results of tests for the various mixture properties when the kneading compaction pressure was varied. Each plotted point is the result of one test, but the good consistency in density trends in Figures 25 and 26 indicates satisfactory molding of each specimen. This offers suitable grounds for establishing, with these limited data, reliable density, stability, and voids trends using kneading compaction of varying intensity as shown in Figures 25, 26 and 27.

The semi-compaction pressure applied at and below compaction pressures of 275 psi was the same as the final compaction pressure. However, at 275 psi compaction pressure and above, a semi-compaction pressure of 275 psi was used for all specimens. This semi-compaction pressure variation did not appear to affect the properties of the compacted samples except for Gradation A. The test values for this gradation at 225 and 275 psi show such a discontinuity that a dashed line has been used to

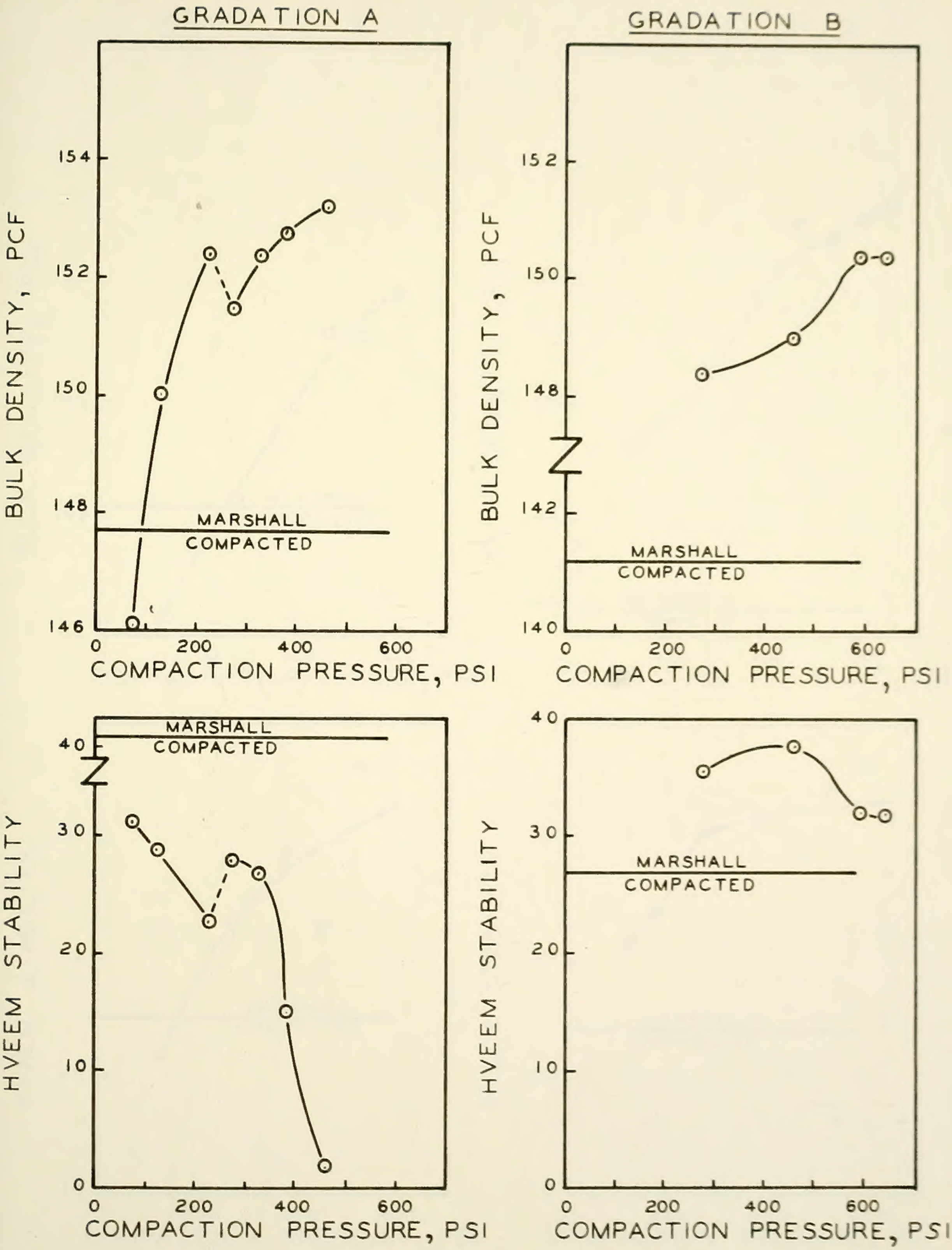

FIG. 25 VARIATION OF HVEEM TEST PROPERTIES WITH COMPACTION PRESSURE - SURFACE MIXTURES

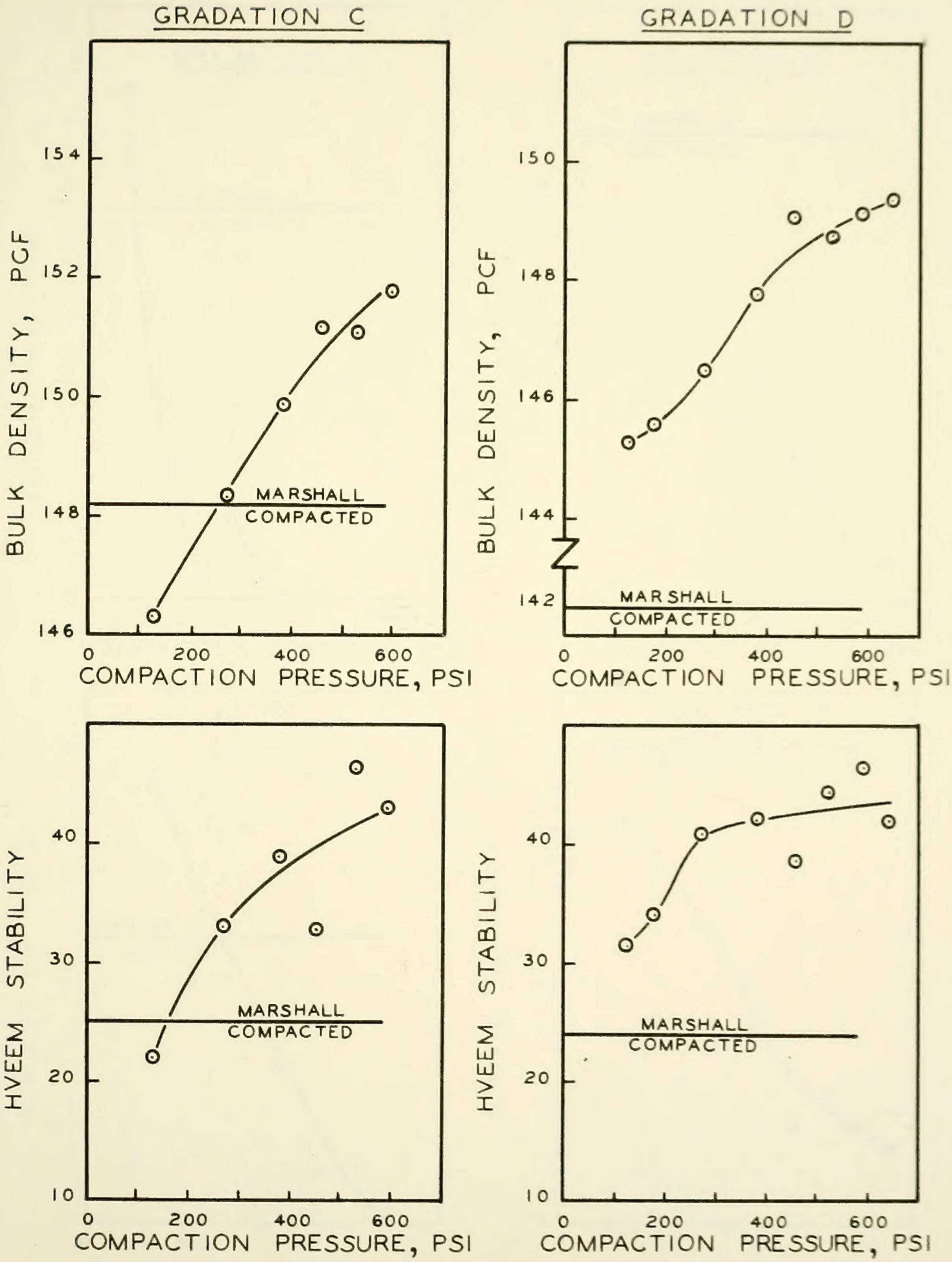

FIG. 26 VARIATION OF HVEEM TEST PROPERTIES WITH COMPACTION PRESSURE - BINDER MIXTURES

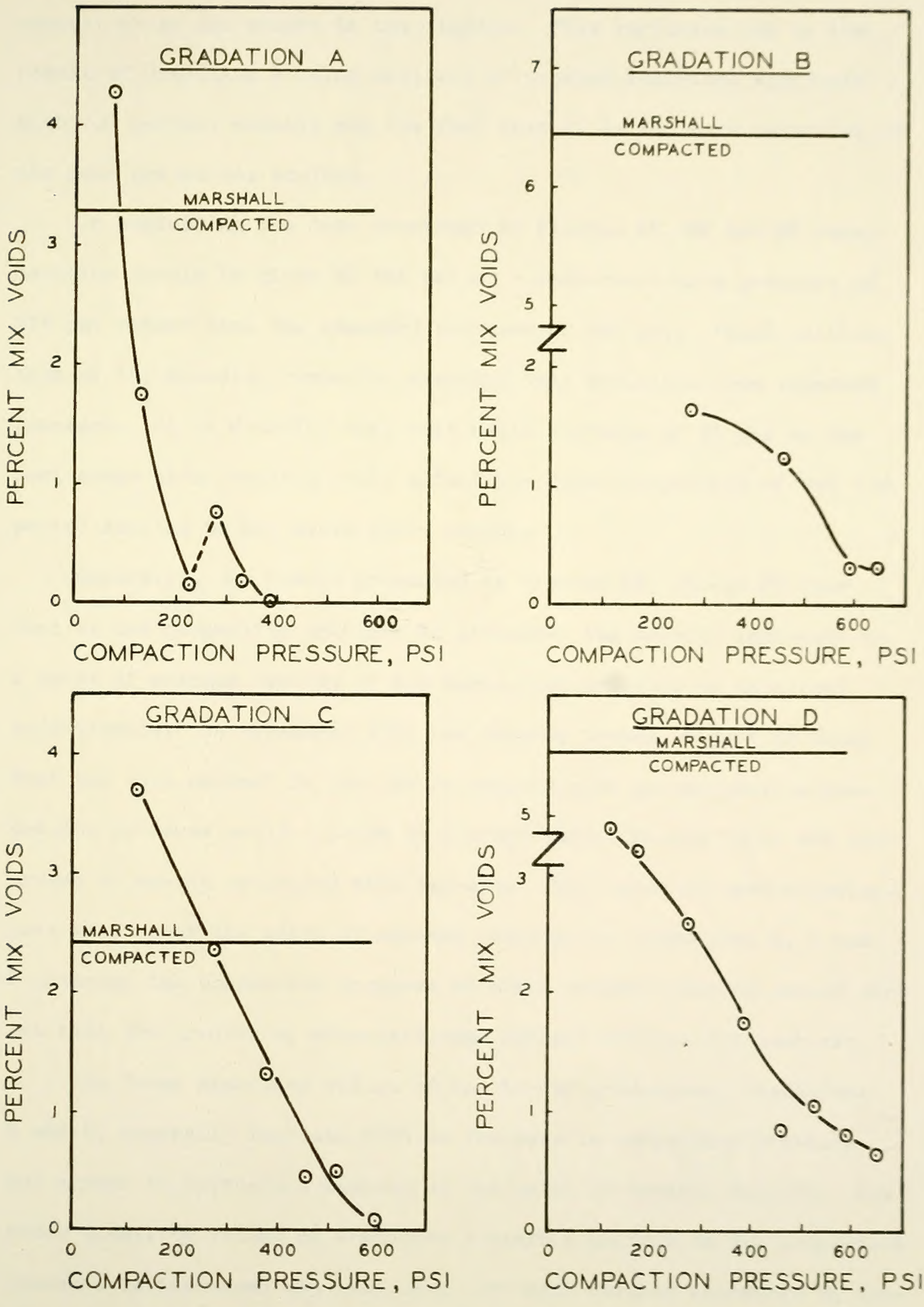

FIG. 27 VARIATION OF VOIDS WITH COMPACTION PRESSURE

connect these two points in the figures. This variation may be the result of Gradation A being entirely of crushed limestone aggregate with 7.0 percent asphalt and the fact that it is the most sensitive of the four gradations studied.

In evaluating the data presented in Figures 25, 26 and 27 consideration should be given to the use of a semi-compaction pressure of 275 psi rather than the standard pressure of 250 psi. Final calibration of the kneading compactor revealed this deviation from standard pressure. It is doubtful that this small increase of 25 psi in the semi-compaction pressure would affect the final properties of the compacted samples to any measureable extent.

Generally, the trends presented by Figures 25, 26 and 27 show that as the compaction pressure is increased the density increases to a point of maximum density if the compaction pressure is increased sufficiently. In agreement with the density trends, Figure 27 shows that the void content in the mix is reduced with an increase in compaction pressure until a point is reached where the mix voids are saturated or nearly saturated with asphalt. This point of saturation occurs at or near the point of maximum density for Gradations B, C and D although the compaction pressure at which maximum density occurs varies with the gradation, materials, and asphalt content for each mix.

The Hveem stability values of the binder gradations, Gradations C and D, generally increase with an increase in compaction pressure but appear to approach a maximum at the point of maximum density. The Hveem stability values of Gradation B reach a maximum as the compaction pressure is increased and then drop off with further increases in compaction pressure. In the case of Gradation A, the Hveem stability de-

creases as compaction pressure increases.

Figure 25 shows that for an increase in density the Hveem stability decreases for each Gradation A test. This same statement can be applied to Gradation B above compaction pressures of about 450 psi, but below this compaction pressure the same trend is produced as for Gradations C and D as shown in Figure 26. In the latter cases the Hveem stability increases with an increase in density.

The standard compaction pressure of 500 psi as used in California would not be feasible or even possible to use for the mixture made from Gradation A. At this pressure the Hveem stability is zero as shown in Figure 25 and the mix voids are completely filled with asphalt as shown in Figure 27. These laboratory test results indicate that the mixture would not pass the standard Hveem stability and voids requirements commonly specified for design in California. However, the performance of this material under non-channelized traffic on U. S. 41 for over five years has been satisfactory. An average field density of 152.5 pcf for the surface mixture at all three sections on U. S. 41 was obtained and the laboratory tests indicate a compaction pressure of approximately 350 psi will give this density. This compaction condition produces a Hveem stability of approximately 24 which is much above the average field stability of 7.7; however, the void content for the laboratory compacted mixture is nearly zero. For this reason the compaction pressure of 300 psi is chosen for the Gradation A mixture as being the most practical compaction pressure to use in order to maintain best agreement between field and laboratory test results for density, Hveem stability and voids.

On the basis of California design procedures, a satisfactory Hveem

stability value of 37 and a void content of nearly one percent are indicated in the figures for Gradation B when a compaction pressure of 500 psi is used. The density obtained in actual pavement service shows that this laboratory compaction pressure of 500 psi is needed to obtain a sufficiently high density assuming that the field and laboratory compaction methods produce the same average specimen density.

California design Hveem stability values of 41 and 42 are given by the data of Figure 26 for mixtures of Gradations C and D, respectively, when a compaction pressure of 500 psi is used. At this compaction pressure void contents are less than one percent for each of these two gradations, as shown in Figure 27. Density values are comparable to those for field samples after one year of service, which is in agreement with the criterion for design used in California.

There is no indication from the data obtained in this study that the standard compaction procedure and criteria used in California could not be applied to Gradations B, C and D. It should be noted that, although the standard California compaction procedure indicates adequate density and Hveem stability values for the mixtures of Gradations B, C and D, the void content of approximately one percent determined by use of the Rice maximum density value is contrary to the desired minimum value of four percent used in California when voids are calculated using apparent specific gravity values for the materials. However, Tables 33 through 36 of Appendix E present apparent specific gravity and Rice specific gravity values for the four mixtures which differ only by 0.01, 0.01, 0.05 and 0.02 for the mixtures of Gradations A, B, C and D, respectively. The apparent specific gravity is higher than the Rice specific gravity for each mixture, and void contents calculated

using the apparent values would not increase above those calculated from the Rice values by more than 0.5 percent for mixtures A and B, 2.3 percent for mixture C and 0.8 percent for mixture D. Therefore, the voids tabulated in this report compare closely to values determined by use of apparent specific gravity values as would be used in California for surface mixtures, and binder voids can be expected to be as much as 2.3 percent lower than the California procedure would indicate. The Gradation A mixture had a low void content of about 0.5 percent at the 300 psi compaction pressure. In order to comply with the four percent voids value used in California, the asphalt content for each of the four mixtures should be reduced. Such a reduction in asphalt content would increase the factor of safety with regard to stability of the mix and somewhat reduce the factor of safety for durability of the mix.

Of particular significance in Figures 25 and 26 is the low Marshall compacted density for each of the four gradations as compared to the density values obtained by kneading compaction. Although the Marshall density was extremely low, the Hveem stability values obtained for these Marshall compacted specimens were above actual service requirements in each case and ranged from 24 to 41. Lower Hveem stability values were obtained for Marshall compacted binder specimens than for surface specimens. Values of 25 and 24 were obtained for Gradations C and D, respectively, and values of 41 and 27 were obtained for Gradations A and B, respectively. Higher Hveem stability values were normally obtained using kneading compaction of nearly any intensity rather than Marshall compaction with the exception of Gradation A. The highest Hveem stability value using kneading compaction for Gradation A was 31 at a compaction pressure of 75 psi and the Hveem stability for Marshall com-

using the measured value would not be those which deviations from the mean values by more than 1.5 percent for mixtures 2 and 5, 2.5 percent for mixture 7 and 3.5 percent for mixture 8. Therefore, the values predicted in this report compare closely to values determined by use of measured specific gravity values as would be used in California procedures, the stability values for [illegible] as much as 4.5 percent lower than the California procedure would indicate. The Predicting a mixture had a low void content [illegible] 500 psi compaction pressure. [illegible] stability value used in California [illegible] four mixtures should be [illegible] would indicate the [illegible] and therefore [illegible]

All compaction [illegible] compacted density [illegible] density values [illegible] mix 8, was extremely low, [illegible] Marshall [illegible] that [illegible] range from [illegible] predicted for Marshall compacted [illegible] mens. Values of [illegible] were obtained [illegible] respectively, and values [illegible] 8, respectively. [illegible] could [illegible] compaction [illegible] Marshall compaction with the exception of [illegible] bility values using Marshall compaction [illegible] compaction pressure of [illegible] Marshall [illegible]

pacted specimens was 41.

Old Spring. The results of tests using the old spring are presented in tabular form in Appendix F and are shown graphically in Appendix G. Figures 40 through 43 of Appendix G can be interpreted in the same manner as Figures 25, 26 and 27 presented for the standard spring in this section. Each point in these figures is an average of three test results. A wide variation of compaction pressures was used which varied for each gradation since pressures were chosen as the work progressed in such a way that satisfactory curves of density and Hveem stability versus compaction pressure could be established.

Differences between the standard and old spring kneading compaction in density and Hveem stability are apparent when the same peak compaction pressures were used and these discrepancies are attributed to the type of action under the compactor foot. The greater impact with the old spring resulted in higher density values at lower pressures. Different particle orientation and more severe degradation are suspected, also, and very possibly are causes of a part of the density and Hveem stability differences. In some cases, using the old spring, an obvious breakdown of aggregate is apparent from the percent mix retained on the No. 6 sieve curves of Figures 40 through 43 shown in Appendix G. Undoubtedly, the high impact was cause of much of this breakage, but no data were taken to determine if similar trends would be obtained using the standard spring.

Figures 44 and 45 of Appendix G picture results for Gradation B and Gradation D, respectively, using the old spring and a constant compaction pressure but with variable compaction time. The compaction

pected stressed (see 8).

Tin Survey. The results of tests using the [illegible] are pre-sented in tabular form in Appendix [illegible] and are shown graphically in Ap-pendix [illegible]. Figures 40 through 43 of Appendix [illegible] the same manner as Figures 1[illegible], [illegible] earlier in this section. Each point in each figure [illegible] three test results, [illegible] white paper for each grade [illegible] in each [illegible] stability [illegible].

Differences between [illegible] in quality and [illegible] particle [illegible] the type of [illegible] the old [illegible] different particle [illegible] and [illegible] (stability differences [illegible] tin [illegible] on the [illegible] Undoubtedly, [illegible] were [illegible] and the [illegible].

Figures 44 and 45 of Appendix [illegible] and [illegible] respectively [illegible] [illegible] pressure [illegible] data. The [illegible]

pressure was high, 595 psi as shown by the kneading compactor calibration in Appendix C (particular reference is made to Figures 37 and 38 of Appendix C), and the mixtures were not sufficiently sensitive in this compaction range to obtain a satisfactory range of density. It is believed that similar curves can be obtained by the use of a standard compaction pressure and variable compaction time as by varying the compaction pressure and using the same compaction time for each pressure. The former method might well be the more desirable approach to use of the two since varying the time of compaction allows more uniform control than varying the compaction pressure and better consistency in the results would be expected with time as the variable. This idea was not pursued any further in this investigation, but relatively few tests with a sensitive mixture in the laboratory would disclose if this approach would be advantageous.

One feature that is definitely illustrated by Figures 44 and 45 is the decrease of Hveem stability as the compaction time is increased beyond the initial point of maximum density. This gives evidence of the importance to avoid a zero-voids condition as maintained by Hveem.

Marshall property results are presented in Figures 46 and 47 of Appendix G for one specimen compacted at each compaction pressure and compacted with the Marshall hammer using 50 blows on each face. Marshall stability trends show no comparison with Hveem stability trends in Figures 40 through 47 of Appendix G, except possibly for Gradation C. For Gradations A, B and D the Marshall stability values increase with an increase in the kneading compaction pressure but the Hveem stability values increase to a maximum for each gradation and, thereafter,

decrease with further increases in compaction pressure. In the case of Gradation C a peak stability is reached at about 500 psi compaction pressure both by the Marshall and Hveem tests, and further increases in compaction pressure reduce the stability values with the decrease in stability being more pronounced by the Hveem test. Marshall-compacted specimens, compared with kneading-compacted specimens, show consistently low Marshall stability and Marshall flow. The low Marshall flow value of 2.4 for Gradation A, using Marshall compaction, is the only unusual property shown by the Marshall data of Figures 46 and 47.

Effects of Kneading Compaction on Specimen Uniformity

Figure 28 illustrates the test results for specimens of each gradation compacted with the standard spring in the compactor. Tabulated results are in Table 38 of Appendix E.

A wide range of density was found to be present in all specimens in which the variation of density was studied. Figure 28 shows that this range is as high as 7 pcf from top to bottom of the Gradation A specimens in the design compaction range and the greatest variation is generally between the middle and bottom layers of the specimen.

It is not believed that these results will correlate well with the actual density variation of a mixture in service. Results are not available in this report to indicate this, however. Cut sections of field and laboratory compacted specimens from this study have shown that the kneading compactor does not produce particle orientation of the same type as produced by construction equipment and traffic in the pavement. In the pavement the particles arrange themselves in a position with the long axis horizontal but the kneading compactor pro-

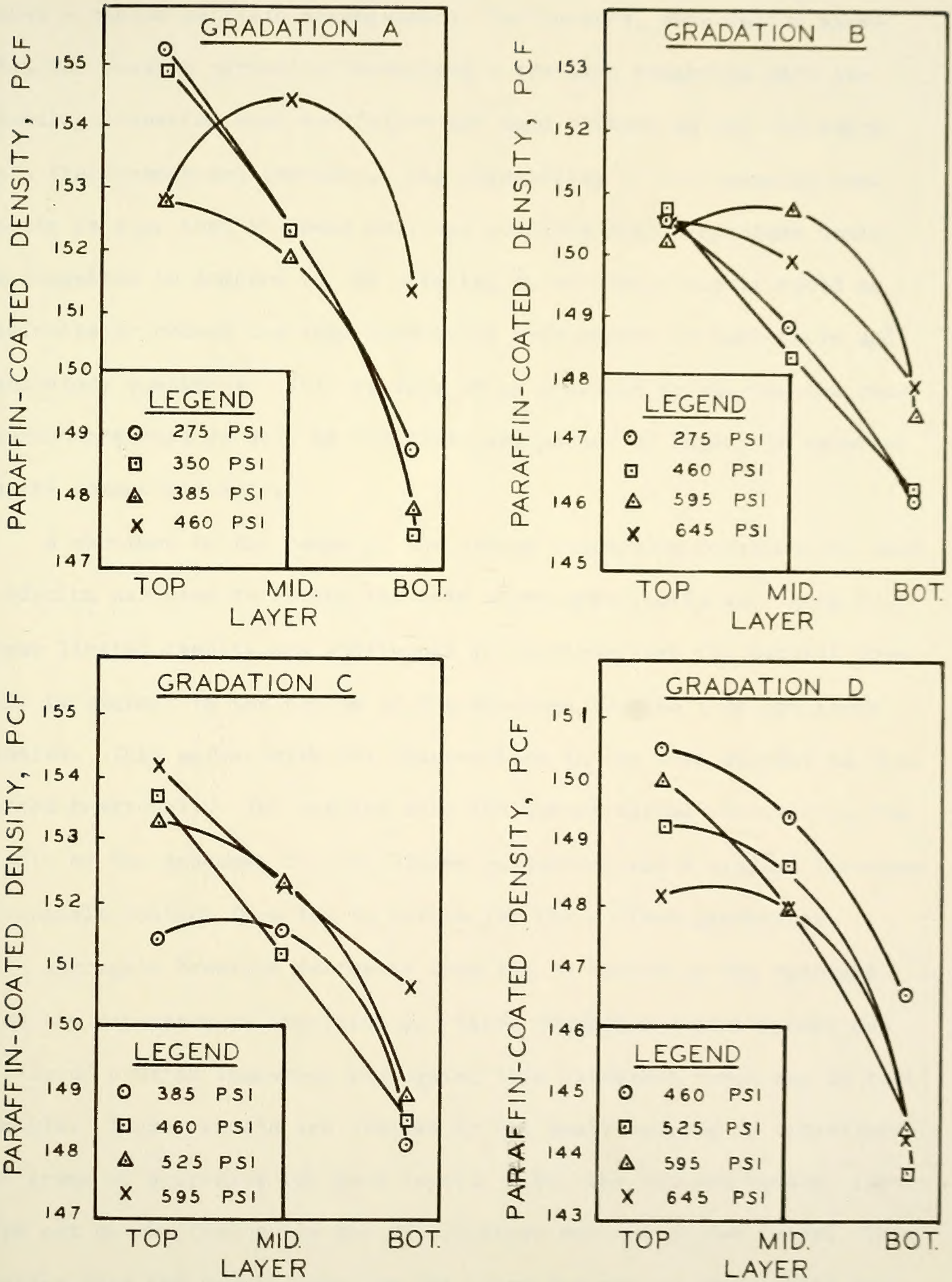

FIG. 28 VARIATION OF DENSITY IN SPECIMENS COMPACTED BY KNEADING COMPACTION

duces a random particle arrangement. Furthermore, observation shows that the density variation throughout a specimen compacted with the kneading compactor does not follow the same pattern as the variation in a field-compacted specimen. The versatility of the kneading compactor is such that it seems entirely possible that a specimen could be compacted to compare to the material in service since it would be desirable to obtain the same structural arrangement in both field and laboratory specimens. This appears to be possible by varying the compaction pressure as well as the depth and number of layers of material in the compaction mold.

A specimen in the range of the design compaction pressure for each gradation was used to obtain the data shown graphically in Figure 29. These limited results are sufficient to conclude that the asphalt content is highest in the bottom of the specimen for the four specimens studied. This agrees with the observations in the core samples as discussed previously. The results show the lowest asphalt content in the middle of the specimen for the binder gradations and a gradual increase of asphalt content from top to bottom for the surface gradations.

Aggregate breakage decreases from top to bottom of the specimen with the exception of Gradation A. Since Gradation A is composed entirely of crushed limestone aggregate, this different trend may be reasonable. These results are limited by the small samples of approximately 400 grams of aggregate for each layer. Also, the top and bottom layers were cut on one face while the middle layer was cut on two faces. This implies that the middle point on the curve may not be established properly with relation to the other two points.

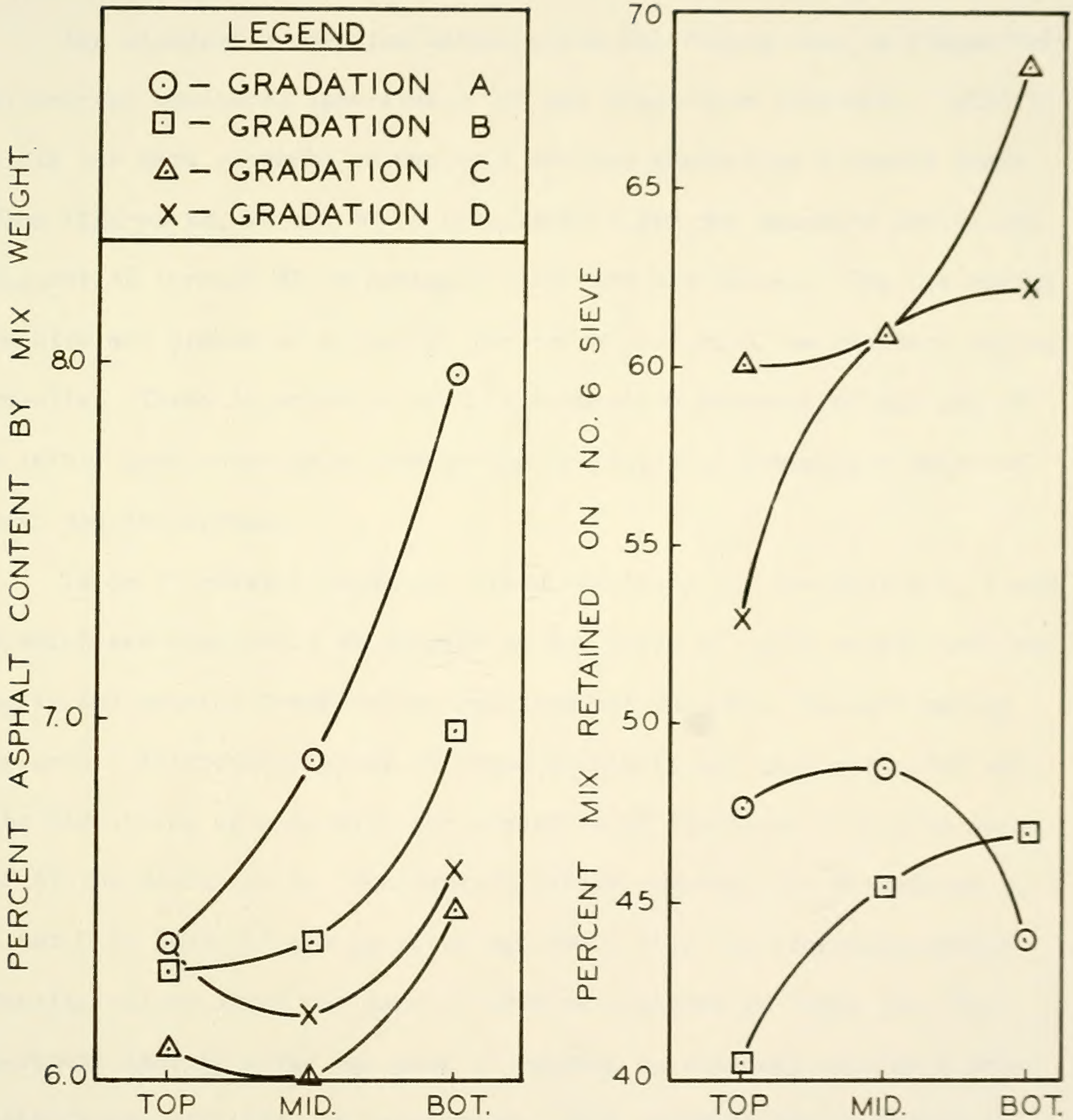

FIG. 29 VARIATION OF PERCENT ASPHALT CONTENT AND PERCENT RETAINED ON NO. 6 SIEVE FOR KNEADING COMPACTED SPECIMENS

Design Compaction Pressures and Design Properties

The standard compaction method which California uses in compacting bituminous specimens specifies a 500 psi compaction pressure. Table 17 lists the test property values at a 500 psi compaction pressure taken from Figures 25, 26 and 27 of this section for the standard spring and Figures 40 through 43 of Appendix G for the old spring. The old spring results are presented primarily for comparison with the standard spring results. There is evidence at this compaction pressure of 500 psi of a rather good comparison between the two types of compaction obtained with the two springs.

Table 17 relates values of Hveem stability for Gradations B, C and D which are completely acceptable on the basis of field sample test results and general Hveem design requirements when the standard spring is used. Acceptable values of Hveem stability are also indicated when the old spring is used with the exception of the Hveem stability value of 31 for Gradation B. The density values reported for Gradations B, C and D in Table 17 are in close agreement with the composite-sample density values after one year of service reported in Table 15. The pavement density after one year of service is commonly used as a primary design criterion in California. Void contents for Gradation B presented in Table 17, when either the standard or old spring is used, are in the range of void contents for individual-layer field samples after five years of service shown in Table 16. No binder void contents were obtained for the field samples but the Table 17 values appear to be lower than service values after five years when referring to composite-sample voids in Table 11. Although these comparisons are made for

Table 17

Test Property Values at 500 psi Kneading Compaction Pressure

Standard Spring.

Gradation	Layer	Highway Pavements Simulated	Bulk Density, pcf	Hveem Stability	Percent Mix Voids
A	surface	41	153.3	0	0.0
B	surface	20, 12	149.3	37	1.1
C	binder	20, 41	151.2	41	0.5
D	binder	12	149.0	42	0.9

Old Spring.

Gradation	Layer	Highway Pavements Simulated	Bulk Density, pcf	Marshall Stability, lbs (Kneading Compacted)	Marshall Flow, 1/100-in. (Kneading Compacted)	Hveem Stability	Percent Mix Voids	Percent Mix Retained on No. 6 Sieve
A	surface	41	153.0	3300	19.2	5 (a)	0.1	49
B	surface	20, 12	149.9	2570	14.6	31	0.7	52
C	binder	20, 41	152.7	3140	21.5	41	0.3	61
D	binder	12	148.0	2460	10.3	40	1.3	60

(a) Extrapolated from the Hveem stability curve of Figure 36 in Appendix G.

field samples taken after five years of service, they are applicable to all field samples taken after one year of service since it has been determined by this investigation that the density achieved after one year of service is comparable to the density after five years of service.

In Table 17 Hveem stability values for Gradations B, C and D are considerably higher than the Hveem stability values of Table 16 after five years of service. This is due to the varying type of compaction between the field and laboratory, and the fact that an average density value is reported when the density actually varies differently throughout the sample depth due the manner of compaction. The laboratory specimen is 2-1/2 in. in height and the pavement was laid in layers varying from one to two inches in thickness. This would account for aggregate particle orientation differences and density differentials which would greatly affect stability results.

By studying the Gradation A curves of Figures 25 and 27 and Figure 40 of Appendix G, a compaction pressure of 300 psi was chosen for compacting the Gradation A mixture in order to obtain satisfactory test properties that would be in agreement with service requirements. Note in Table 17 for the old spring that the highest Marshall stability for the four gradations is obtained for Gradation A when kneading compaction is used and the lowest Hveem stability value of the four gradations is also indicated for Gradation A. This implies that a mixture may have a void content near zero and yet have a high Marshall stability value. This is in contrast to both what is maintained by Hveem as being a desired stability-voids relation and to what is shown by field-performance data as being a desired condition. The final test

properties are listed in Table 18 as indicated by the data at the design compaction pressure chosen for each mixture.

From the discussion to this point it appears that the normal 500 psi compaction pressure can be used for Indiana binder mixtures, but a lower design compaction pressure should be employed for compacting more sensitive mixtures. These sensitive mixtures are usually surface mixtures, especially those surface mixtures containing high percentages of crushed limestone aggregate and a high asphalt content.

A low Hveem stability value seems to establish readily if a mix is over-compacted at a set asphalt content. If a condition of failure is present, it will be indicated by a low Hveem stability value resulting from a near-zero void content due to an excess of asphalt in the mixture. It is thought that if the Hveem stability is above a value of 25 at a compaction pressure of 500 psi, there is no need to lower the design asphalt content provided the design criteria of voids and density are satisfied. This statement is one possible interpretation which may be drawn from the California criterion of establishing an absolute minimum Hveem stability value of 25 under any conditions. Service Hveem stability requirements presented in this report also show that 25 would be an approximate minimum desirable Hveem stability to be applied to all mixtures under all conditions of traffic; however, the broad assumption is made for this statement that actual pavement stability can be directly related to the Hveem stability of laboratory compacted samples.

Tables 19 and 20 record comparisons of aggregate gradations to show degradation effects. These data show no significant increase in degradation after 1957 (three years of service), and no data are pre-

Table 18

Test Property Values at Design Compaction Pressures

Standard Spring.

Gradation	Layer	Highway Pavements Simulated	Compaction Pressure, psi	Bulk Density, pcf	Hveem Stability	Percent Mix Voids
A	surface	41	300	152.0	28	0.4
B	surface	20, 12	500	149.3	37	1.1
C	binder	20, 41	500	151.2	41	0.5
D	binder	12	500	149.0	42	0.9

Old Spring.

Gradation	Layer	Highway Pavements Simulated	Compaction Pressure, psi	Bulk Density, pcf	Marshall Stability, lbs (Kneading Compacted)	Marshall Flow, 1/100-in. (Kneading Compacted)	Hveem Stability	Percent Mix Voids	Percent Mix Retained on No. 6 Sieve
A	surface	41	300	150.9	2680	19.9	36	1.1	49
B	surface	20, 12	500	149.9	2570	14.6	31	0.7	52
C	binder	20, 41	500	152.7	3140	21.5	41	0.3	61
D	binder	12	500	148.0	2460	10.6	40	1.3	60

Table 19

Aggregate Gradation Comparisons - Surface (a)

Sieve Size Pass.	Sieve Size Ret.	Specification Limits, Percent	Percent Fuller's Gradation	1954(b)	1957(c)	1959(d)	
Gradation A							At 285 psi (f)
1/2"	3/8"	2 - 14	13.4	10.8	8.7	7.0	9.5
3/8"	#4	20 - 50	25.4	36.5	33.4	34.9	28.5
#4	#6	0 - 11	9.8	8.8	9.5(e)	10.1	14.4
#6	#8	0 - 11	8.1	4.1	6.6	6.1	5.6
#8	#16	5 - 20	12.6	10.5	10.6	11.0	10.3
#16	#50	10 - 25	15.4	16.1	15.9	16.9	17.9
#50	#100	2 - 17	4.4	4.1	4.5	4.9	4.3
#100	#200	1 - 5	3.3	3.3	3.3	2.5	3.5
#200	---	3 - 5	7.6	5.8	7.5	6.6	6.0
Mix Retained on #6		45 - 55	---	52.2	48.1	48.3	48.7
Gradation B							At 525 psi (f)
1/2"	3/8"	2 - 14	13.4	9.2	13.0	6.6	9.1
3/8"	#4	20 - 50	25.4	39.9	35.3	39.5	33.9
#4	#6	0 - 11	9.8	9.6	8.9(e)	10.0	11.8
#6	#8	0 - 11	8.1	3.8	6.1	5.3	5.1
#8	#16	5 - 20	12.6	5.4	5.8	9.0	7.9
#16	#50	10 - 25	15.4	24.1	21.5	19.4	22.4
#50	#100	2 - 17	4.4	4.8	5.3	5.4	5.1
#100	#200	1 - 5	3.3	0.6	0.7	0.9	1.3
#200	---	3 - 5	7.6	2.6	3.4	3.9	3.4
Mix Retained on #6		45 - 55	---	55.2	53.8	52.7	51.5

(a) All aggregate is from asphalt-extracted samples.

(b) Average of three to eleven samples after mixing and before rolling.

(c) Samples taken from between-wheeltrack position (average of three).

(d) Samples taken from wheeltrack position (average of three to five).

(e) Interpolated on the No. 6 sieve since a No. 6 sieve was not used in the sieve series.

(f) Average for three samples compacted with old spring.

Table 20

Aggregate Gradation Comparisons - Binder (a)

Sieve Size		Specification Limits, Percent	Percent				
Pass.	Ret.		Fuller's Gradation	1954(b)	1957(c)	1959(d)	At 525 psi (f)
Gradation C							
1"	1/2"	5 - 50	29.3	41.5	24.3	29.0	24.9
1/2"	#4	10 - 60	27.4	26.7	38.0	35.5	36.6
#4	#6	0 - 5	7.0	1.5	3.7(e)	3.4	3.7
#6	#8	0 - 5	5.6	2.0	2.6	2.5	3.1
#8	#16	3 - 12	9.1	6.7	7.5	7.5	7.3
#16	#50	5 - 20	10.8	10.7	11.1	10.7	12.3
#50	#100	2 - 10	3.1	3.1	3.5	3.6	3.0
#100	#200	0 - 4	2.3	2.5	2.9	2.2	3.7
#200	---	0 - 4	5.4	5.3	6.4	5.6	5.4
Mix Retained on #6		60 - 70	---	65.8	62.0	64.0	61.5
Gradation D							
3/4"	1/2"	5 - 50	18.3	20.1	10.4	8.6	13.6
1/2"	#4	10 - 60	31.7	42.7	49.3	47.9	43.2
#4	#6	0 - 5	8.0	5.4	6.3(e)	6.7	6.2
#6	#8	0 - 5	6.6	2.7	4.4	4.1	3.7
#8	#16	3 - 12	10.4	4.5	4.7	5.8	6.2
#16	#50	5 - 20	12.5	20.0	18.0	19.8	19.7
#50	#100	2 - 10	3.6	3.6	4.1	4.4	3.9
#100	#200	0 - 4	2.7	0.4	0.6	0.7	1.2
#200	---	0 - 4	6.2	0.6	2.2	2.0	2.3
Mix Retained on #6		60 - 70	---	64.1	62.6	59.3	59.2

(a) All aggregate is from asphalt-extracted samples.

(b) Average of three to eleven samples after mixing and before rolling.

(c) Samples taken from between-wheeltrack position (average of three).

(d) Samples taken from wheeltrack position (average of three to five).

(e) Interpolated on the No. 6 sieve since a No. 6 sieve was not used in the sieve series.

(f) Average of three samples compacted with old spring.

sented to give evidence of aggregate breakage occurring under traffic. However, results for samples taken immediately after initial field compaction, are needed in order to draw conclusions in regard to degradation effects of traffic after three and five years of service.

The gradation in the final column of each table is from the results obtained with kneading compaction nearest the design-compaction pressure. These results are from samples compacted with the old spring and it is not known how nearly they would represent the final aggregate gradation if the standard spring had been used. In general, the laboratory-compacted gradation is near to either the 1957 or 1959 field gradation with the largest deviations in the two coarse aggregate fractions above the No. 4 sieve size for each gradation. On the average, the laboratory-compacted gradation does not differ from the 1957 or 1959 field gradation by more than one to two percent for any fraction.

Supplementary Results

During the progress of the study several interesting possibilities or effects of the kneading-type compaction became apparent. The results of three such individual studies are discussed in this section and presented to add to the information otherwise obtained in this investigation.

Remolded Samples

The recompacting of a field sample in the laboratory to reproduce field density and stability was attempted. The old spring was installed in the compactor at this time, but the same trends would be expected if the standard spring had been used.

The remolded test results are tabulated in Table 39 of Appendix F. The upper portion of Table 39 consists of remolded test results of density and Hveem stability for the same materials in each specimen as was used for the original tests of density and Hveem stability on the pavement cores. These results are analyzed statistically in Part B of Appendix H, and data of the core and remolded results are presented there, and also in Table 21, for ready comparisons. The statistical analysis shows that the density for remolded surface samples at 595 psi compaction pressure was not significantly different from the original density for Gradation B. Although there is no statistical difference, it is suspected that the difference may be significant technically; that is, the small density variation may have a large effect on stability. The density difference was highly significant for the two binder gradations as shown by the statistical comparisons in Part B of Appendix H. In Table 21, as in Part B of Appendix H, the Hveem stability values show the reverse trend of the trend shown for density. The surface stability is significantly different and the binder stability is not significantly different. These conclusions defintely show a difference in the structure of the field and laboratory specimens, or little correlation between field and laboratory compaction. These conclusions leave a desire to check the results using the standard spring before definitely stating that the remolding method has much true meaning or value.

As the remolded samples were being compacted it was observed that the aggregate did not tend to degrade seriously. This follows Hveem's conclusion that the mixture approaches a maximum density under heavy

Table 21

Comparison of Pavement Core and Recompacted Specimen Test Results (a)

Layer	Specimen Identification	Corresponding Laboratory Gradation	Bulk Density, pcf		Hveem Stability	
			Core	Remolded	Core	Remolded
surface	20-0A	B	153.2	153.4	34.8	16.2
surface	20-0B	B	151.5	152.4	23.2	4.4(b)
surface	20-0C	B	150.8	153.0	28.9	5.7(b)
surface	12-0A	B	152.3	154.6	34.0	17.7
binder	20-0A	C	150.2	155.8	29.0	25.6
binder	20-0B	C	147.2	154.4	22.2	35.8
binder	20-0C	C	---	---	23.4	24.0
binder	41-0A	C	150.6	155.4	48.7	40.1
binder	41-0B	C	149.2	154.8	27.8	25.0
binder	41-0C	C	149.0	153.3	35.4	29.2
binder	12-0A	D	151.8	152.0	36.6	20.2
binder	12-0B	D	150.9	153.3	15.2	14.5
binder	12-0C	D	148.4	155.1	22.4	39.6

(a) Specimens remolded from tested pavement cores.

(b) Results are extrapolated when the lateral pressure exceeded the maximum dial reading of 200 psi.

traffic conditions. The gradations in this study tend to approach a maximum density, or an aggregate gradation close to Fuller's curve as shown by the data of Tables 19 and 20. This trend is more predominant for Gradations A and C which are composed entirely of crushed limestone aggregate. This would explain why very little breakage was observed when recompacting the material, and reason exists to believe that field conditions can be duplicated by remolding if the versatility of the kneading compactor is employed to greater advantage.

Comparison of Hveem and Marshall Stabilities Using Kneading Compaction

Results are shown in Figure 48 of Appendix G comparing Hveem and Marshall stability results for each of the four mixtures. Each point is plotted by averaging three test results for the Hveem stability value and one test result for the Marshall stability value for kneading compacted samples.

Again, the data were obtained with the old spring and it is doubtful that similar results would be obtained with the standard spring in the normal range of compaction. The low Hveem stability value at the high compaction pressure for each gradation is believed to result from high aggregate breakdown and asphalt "flushing" in the upper 1/4 inch of the specimen. This is caused by the high impact from the spring being over-loaded and results in a very low surface stability. Such an impact action was not observed when compacting with the standard spring. However, it does seem that as the compaction pressure is increased, a peak Hveem stability value will be reached and below this any Hveem stability will represent two Marshall stability values.

The pressure at which the peak Hveem stability is obtained may be very high for the standard spring but there is reason to believe that such a pressure would be reached.

Aggregate Degradation

Aggregate degradation has been studied in a very general way in this investigation in order to establish the definite need for further study in this area. The data clearly illustrate that aggregate breakage occurs in bituminous pavements either by rolling at the time of construction or under service traffic or by a combination of the two. Some qualitative values are also presented in the form of gradation comparisons to indicate the magnitude of any degradation. (Refer to Tables 19 and 20 of this section and Tables 42, 45, 48 and 51 of Appendix F).

The gradations used are open in type and they do not originally follow Fuller's maximum density curve; however, after three years of service the gradations seem to approach a gradation comparable to Fuller's gradation. This point may be reached prior to three years of service, but data are not available here to substantiate this.

The laboratory data relating to aggregate degradation in this study was obtained with the old (low-capacity) spring in the compactor and the breakage of aggregate obtained in the various sieve fractions correlates well with that obtained after three or five years of service. Gradation A presents several trends that differ radically from the trends presented for Gradations B, C and D. This is due, probably, to use of crushed limestone fine and coarse aggregate and a high asphalt content of 7.0 percent in the Gradation A mix. Gradation C is also

composed entirely of crushed limestone aggregate, but with 5.7 percent asphalt this binder gradation apparently produced a stable mix for a wide range of compaction pressures. All of the gradations use a crushed limestone coarse aggregate from the same source which poses a restriction on the data reported and on conclusions drawn from these data.

The aggregate gradations for laboratory-compacted specimens show a tendency to approach Fuller's gradation as the data of Tables 19 and 20 indicate. Gradations A and C are composed entirely of crushed limestone aggregate and the gradation results show that this aggregate was degraded both by laboratory compaction and in the field to closely approach the Fuller gradation. The overall gradation of aggregate samples from Marshall-compacted specimens was found to be closer to the three or five-year service gradations than any results using kneading compaction for all of the compaction pressures applied.

Evaluation of the effect of kneading compaction on aggregate degradation was attempted by comparing the laboratory compacted gradations with the five-year service gradation. The average percent difference retained between each sieve group was found using the data of Tables 42, 45, 48 and 51 of Appendix F and it was concluded that no trend exists over the range of compaction pressures used. This conclusion is also apparent in the gradation data presented in the Appendix F tables. Indications are that maximum breakage occurs at very low pressures of less than 50 or 100 psi and, above this pressure, no significant degradation increase will occur. Most commonly, the difference of any percent between sieves for the laboratory-compacted aggregate, compared to the desired (five-year service) value, was between one and two percent.

SUMMARY OF RESULTS

The following statements are presented as a resume of the findings resulting from this investigation. The field results are presented as being representative of the three pavements studied and presumably can be applied to resurface pavements of similar construction and service characteristics. The laboratory testing has been restricted to mixtures which simulate the mixtures used in service and the laboratory test results are applicable only in cases where techniques and procedures do not differ radically from those employed for this project.

Composite Pavement Samples

1. Test results of density and Marshall tests for composite samples from four samplings during a five-year period have shown that the traffic-compacted density and Marshall stability reached values near maximum after one year of service. This study has shown that traffic usually increased density values about 6 pcf after construction, although the range was 2 to 13 pcf since the tests for the as-constructed condition showed random and variable results, and the Marshall stability approximately doubled as a result of the density increase. For any period after one year, density was consistently highest at traffic-signal intersections when compared with the density at sections under more uniform traffic flow. The range of this density difference was about zero to 6 pcf for samples tested in this study, and a difference of 3 pcf was most common. As a result of the increased density at inter-

sections, the Marshall stability at intersections for any period after one year of service was commonly about 150 percent of the Marshall stability under more uniform traffic flow.

2. Density and Marshall stability results showed a wide variation with respect to the two positions under channelized and heavy traffic on U. S. 12, but no significant difference between positions was indicated under non-channelized traffic and lower traffic volumes as on U. S. 20 and U. S. 41. A uniform pattern with respect to section was apparent from the density and Marshall stability results for samples from channelized traffic lanes on U. S. 12, but a non-consistent pattern was obtained for samples taken from non-channelized traffic lanes on U.S. 20 and U. S. 41.

3. Marshall flow values did not exceed the as-constructed flow values throughout the five-year service period and they were lower than as-constructed values for several cases. Flow values, on the average, did not vary significantly with section on U. S. 20 and U.S. 12, but on U. S. 41 the flow values tended to increase with progressive increases in distance from the intersection. The above statements are applicable to samples from both positions, wheeltrack and between-wheeltracks, and generally flow values for the two positions did not differ by more than five with the highest values in the wheeltrack position for about two-thirds of all cases.

4. All Hveem stability values for composite field samples were less than 35, or lower than the minimum permissible laboratory compacted Hveem stability value used for design in California. On U. S. 41 the Hveem stability for composite samples at 500 feet and 1000 feet from the intersection was nearly the same and the Hveem stability at the

intersection was about 150 percent of the values at the other two sections. All Hveem stability values for U. S. 41 were higher for between-wheeltrack samples than for wheeltrack samples by about 20 percent. Hveem stability values for U. S. 20 samples were all near to 24 regardless of section or position for the limited number of test samples. For U. S. 12 composite samples from the between-wheeltrack positions were about 28 and the only wheeltrack sample test result was 13 for a sample at the intersection where rutting had become serious.

5. Composite-sample void contents were found to be about two percent at intersections and four percent under more uniform traffic flow when the Rice maximum density values were used to compute void contents.

<u>Individual-Course Pavement Samples</u>

1. All Hveem stability values on surface samples were less than 35 and the range was from 5.4 to 34.8. Only 20 percent of the stability values on binder samples were over 35 and the range was from 15.2 to 48.7.

2. The surface density was nearly the same for the two positions, wheeltrack or between-wheeltrack, at any one section of a pavement; however, the binder density varied with position and did so more for conditions of traffic on U. S. 20 and U. S. 41 than for the U. S. 12 condition, with the exception of section C on U. S. 12.

3. A maximum of two percent voids was obtained for between-wheel track surface samples and an average value, using samples at the intersection and 1,000 feet from the intersection, of about one percent was indicated for each pavement. The voids for between-wheeltrack

interaction was about 50% percent of the values in the [illegible] sta-
tions. Although normality [illegible] were higher [illegible]
[illegible] samples than for [illegible] samples [illegible]
[illegible]
[illegible]
[illegible]
[illegible]
[illegible]

1. [illegible]
percent [illegible]
[illegible]
[illegible].

[illegible]

1. All [illegible]
[illegible]
[illegible]
[illegible]

2. The [illegible]
[illegible]
[illegible], the [illegible]
[illegible]
[illegible]

3. [illegible]
[illegible]
[illegible]
[illegible]

samples should also indicate voids for wheeltrack samples since density values for the two positions are comparable for all cases. No voids for binder samples were determined but results for surface and composite samples indicated a binder void content of near zero to six percent depending on the section. Binder voids at the intersection would seem to normally be three percent but in some cases, as on U. S. 12, the composite sample results indicated binder voids were equivalent to surface voids at the intersection. Normally binder voids were higher for between-wheeltrack samples.

4. Extracted aggregate samples from the pavements showed degradation primarily of the plus No. 4 sieve size of coarse aggregate and a definite increase in the minus No. 200 sieve material for all cases of surface and binder samples.

Laboratory-Compacted Samples

1. The Centrifuge Kerosene Equivalent and Oil Equivalent procedures indicated optimum asphalt contents by mix weight of 5.5 percent for Gradations A, C and D and 5.2 percent for Gradation B. Gradations A and B are surface gradations complying with specifications for Indiana type B and Gradations C and D are Indiana binder gradations. Gradation A was used on U. S. 41, Gradation B on U. S. 20 and U. S. 12, Gradation C on U. S. 20 and U. S. 41, and Gradation D on U. S. 12.

2. Variation of the kneading compaction pressure for the four mixtures used in this study showed that density increased as the compaction pressure was increased. A point of maximum density was approached or obtained for all four mixtures when the compaction pressure was increased sufficiently to a point where the voids were saturated,

or nearly saturated, with asphalt. Hveem stability values of the binder mixtures (C and D) generally increased with an increase in compaction pressure and appeared to approach a maximum near maximum density. For these binder mixtures an increase in density increased the Hveem stability. The Hveem stability for surface mixture B reached a maximum with increased compaction pressure and, thereafter, decreased with increased compaction pressure. For surface mixture A an increase in compaction pressure decreased the Hveem stability, as for mixture B after reaching a maximum stability. Thus, an increase in density was accompanied by a decrease in Hveem stability.

3. Marshall-compacted densities for each of the four mixtures were much lower than kneading-compacted densities, but Hveem stability values for Marshall-compacted specimens were well above those for pavement-core samples for each mixture. Higher Hveem stability values were obtained using kneading compaction of nearly any intensity rather than Marshall compaction except for Gradation A which gave a higher Hveem stability for Marshall compaction than for any kneading-compaction pressure.

4. Specimen uniformity tests showed that the density variation was as high as 7 pcf between top and bottom layers of a kneading-compacted specimen with the bottom of the specimen being the least dense. A larger density difference usually existed between the middle and bottom layers than between the top and middle layers. Asphalt contents were highest in the bottom layer for four specimens (one for each mixture in the design compaction pressure range) and aggregate degradation was greatest in the top of each specimen except for Gradation A where degradation was greatest in the bottom layer.

5. Several Gradation B and D specimens compacted at 595 psi with the old spring and for various compaction times showed that the Hveem stability decreased after the compactive effort was increased beyond the initial point of maximum density.

6. Using kneading-compacted specimens compacted with varying pressures with the old spring, the Marshall stability trends show no comparison with Hveem stability trends, except possibly for Gradation C. For increases in compaction pressure Hveem stability values increased to a maximum and, thereafter, decreased. Marshall stability values increased with increased compaction pressures for all pressures used except for Gradation C which gave a maximum Marshall stability at 500 psi compaction pressure and then decreased. Plots of Hveem stability versus Marshall stability show a peak, below which any two specimens may have the same Hveem stability but different Marshall stabilities. Marshall-compacted specimens, compared with kneading-compacted specimens, show consistently low Marshall stability and Marshall flow values.

7. Density and Hveem stability values for remolded specimens using the old spring and 595 psi compaction pressure resulted in surface density values comparable to the surface density for pavement samples, and binder density values significantly higher than the binder density for pavement samples. A reverse trend was obtained in relation to the Hveem stability values with comparable results obtained for the binder mixtures and significantly low results obtained for the surface mixtures after remolding.

CONCLUSIONS

The problem and need of simulating field conditions in the laboratory are presented by this investigation. The all-important task is to develop a method and procedure for compaction that will produce a specimen that represents the mixture, including structural arrangement, as it will be in service. To solve this problem a knowledge of the basic qualities of the aggregate and asphalt properties is essential. Viscous resistance of the asphalt is a factor and known to be steadily overcome as the loading time increases. The aggregate hardness, durability, and resistance to impact and abrasion are also factors to be considered.

The results of this investigation give evidence that the kneading compactor does not produce a compacted bituminous specimen having the same physical characteristics as the material after construction and traffic compaction in the field under Indiana conditions. Cut sections of field and laboratory compacted specimens from this study have shown that the kneading compactor does not produce particle orientation of the same type as produced by construction equipment and traffic in the pavement. In the pavement the particles arrange themselves in a position with the long axis horizontal, but the kneading compactor produces a random particle arrangement. Furthermore, observation shows that the density variation throughout a specimen using standard compaction with the kneading compactor does not follow the same pattern as the variation in a field compacted specimen. Evidence is given of this density

differential by the wide variation between field and laboratory Hveem stability values obtained for specimens having comparable average densities. It is important to consider the above comments as being applicable only to resurface pavements and mixtures studied for this investigation. For different materials, construction procedures and traffic conditions other than those studied for this work the kneading compactor might closely approach the field compaction condition. The kneading compactor is recognized as one of the most reliable methods of compaction now in common use; however, the point to be emphasized is that although it simulates field compaction to a higher degree than earlier types of compaction used in the laboratory, there is a need to improve the compaction technique to reproduce the field condition better for Indiana conditions.

In addition to the above comments, the following statements are presented as a resume of the conclusions resulting from this study.

1. Performance studies have indicated that the overall performance of the three highway pavements investigated is satisfactory after five years of service. Only at the traffic-signal intersection on U. S. 12 is the pavement approaching a failure condition shown by rutting and shoving of the resurface pavement. Traffic in this zone is channelized and consists of a very high volume of heavy trucks.

2. Sampling techniques employed for this study appear to be satisfactory; however, it would be desirable to have a larger pavement sample with less cut aggregate to be used for tests other than the Hveem stability test. In order to obtain more reliable and complete field data it would be necessary to obtain test results for samples of "production mixtures" taken at the plant while the project is in

progress. This procedure would give test values for the mixture which was actually used in construction and it would eliminate the error involved when attempts are made to reproduce field mixtures in the laboratory.

3. Laboratory techniques used in this study for general testing of the core samples are believed to be adequate since the nature of the sample in most cases does not warrant more precise testing. It is recommended that built-up specimens of pavement samples for Hveem stability tests have no single layer thickness less than the maximum aggregate size and that the number of layers be kept to a minimum.

4. Asphalt extraction results on pavement samples led to the conclusion that asphalt is probably being forced from the surface into the less-dense binder as traffic continues to densify the resurface pavement. Results indicate that this effect becomes more pronounced with time. Increased asphalt contents also were found for the less-dense area of laboratory compacted specimens and generally this condition is thought to be present only as long as the void content of the less-dense portion of the specimen is high as in an open-type mixture.

5. To establish a laboratory design procedure based on service requirements it would ultimately be desirable to consider the varying effects of traffic. A method which does not consider traffic variation consists of averaging the wheeltrack data for all sections, as was done in this study, when sufficient data were not available to consider each section individually. A design procedure based on average data will result in an over-design condition for pavements subjected to uniform traffic and an under-design condition at intersections where failure problems will probably result in a proportionally short time. To

overcome the problem of traffic variation a possible design approach would be to vary the Stabilometer test-speed to represent various traffic conditions. Ultimately the same Hveem stability might be required for all traffic conditions, but a slow test speed would indicate lower stability values than a faster test speed when a constant mixture is used. This would indicate the use of a more stable mixture for the more severe traffic conditions as at intersections.

6. Hveem stability values as low as 7.7 for pavement samples of the surface layer show satisfactory performance under high-volume traffic when traffic is non-channelized. It is believed that this would probably not be true for the more severe conditions of very high volume and channelized traffic as on U. S. 12. On U. S. 12, sections were showing failure when Hveem stability values for composite samples were averaging 13.1. This indicates that a somewhat more stable mixture is required for these sections of U. S. 12 but data were not available from this study to indicate the magnitude of this required stability increase. Binder Hveem stability values were as low as 15.2 on U. S. 41 where satisfactory performance was shown. Normally, values of Hveem stability on U. S. 20 and U. S. 41 were about 25 but they ranged from 15.2 to 36.6. On U. S. 12 failure was shown at the intersection where the Hveem stability value was 24.0 in the wheel-track position and 48.7 in the between-wheeltrack position. These same two values are the minimum and maximum stability values for the U. S. 12 pavement. It is concluded that a Hveem stability of less than 24 would not show satisfactory performance at the intersection for this high-volume traffic condition.

7. Consistency tests of penetration, softening point, and ductility on the recovered asphalt from surface samples indicate that no unusual problems exist with the asphaltic material in service, and no relationship was found between asphalt properties and pavement performance.

8. Although a Hveem design series was not made for each of the four laboratory mixtures used in this study, it is obvious that the Hveem method would indicate a lower optimum asphalt content than those actually used. This is verified somewhat by the CKE and OE data for each gradation which indicate optimum asphalt contents of 5.5 percent by weight of mix for surface Gradation A and binder Gradations C and D and 5.2 percent for surface Gradation B. Gradation A was used on U. S. 41, Gradation B on U. S. 20 and U. S. 12, Gradation C on U.S. 20 and U.S. 41, and Gradation D on U. S. 12.

9. Tests on laboratory mixtures show that the standard compaction pressure of 500 psi as used in California would not be feasible or even possible for the Gradation A mixture since the Hveem stability is zero and the mix voids are completely filled with asphalt even when lower pressures are used. It is concluded that the compaction pressure for this mixture would have to be reduced to 300 psi to obtain a void-density relationship approximately equal to that for pavement samples. An alternate procedure to increase the void content would be to decrease the asphalt content which would increase the mix stability qualities but decrease the durability qualities. The void contents for mixtures of Gradations B, C and D could also be increased to agree better with accepted voids in California (where a minimum

of four percent voids is preferred) by using a lower asphalt content, although there is no indication by the stability results that the standard California compaction procedure cannot be applied to these three mixtures. These three mixtures also show density values in close agreement with the composite-sample density values after one year of service, which is used in California as a primary design criterion. Generally, the normal 500 psi compaction pressure can be used for Indiana binder mixtures, but a lower design compaction pressure should be employed for compacting more sensitive mixtures. These sensitive mixtures are usually surface mixtures, especially those surface mixtures containing high percentages of crushed limestone aggregate and a high asphalt content.

10. The Rice specific gravity values for the four laboratory mixtures and mixture specific gravity values calculated from apparent specific gravity values generally agree closely and void contents calculated by the two methods would not differ by more than 0.5 percent for the surface mixtures and 0.7 percent for the Gradation D mixture. For the Gradation C mixture differences as high as 2.3 percent are possible and this difference would be considered significant. With this information available the voids in this report using Rice specific gravity values are comparable with voids by the California procedure using apparent specific gravity values, except for mixture C voids.

11. When a bituminous mixture has a near-zero void content, pavement performance shows that the continued compactive effort of traffic will decrease stability. This trend is also indicated by laboratory Hveem stability values which decrease rapidly with further

compaction of the mixture after a near-saturated condition is reached. Contrary to Hveem stability results and pavement performance, Marshall tests generally show that the stability increases with increased compaction over the wide range used in this study. Only in one case did the Marshall stability decrease with an increased compactive effort and this decrease was not as pronounced as that obtained by the Hveem test. On this basis, it is concluded that the Hveem stability results do show some correlation with pavement performance for most cases and the Marshall stability test results show little correlation with pavement performance. Furthermore, it is thought that if the Hveem stability is above a value of 25 at a compaction pressure of 500 psi, there is no need to lower the design asphalt content provided the void-density relationship in the laboratory is comparable to the field relationship. This is because a serious condition of failure will normally be indicated by a Hveem stability value lower than 25.

12. Differences between density and Hveem stability obtained by standard and old-spring kneading compaction are apparent when the same peak compaction pressures are used and these discrepancies are attributed to the type of action under the compactor foot. The greater impact with the old spring resulted in higher density values at lower pressures. Different particle orientation and more severe degradation are suspected, also, and very possibly are causes of a part of the density and Hveem stability differences.

13. Statistical data for pavement samples justify concluding that the Stabilometer test does not significantly affect the specimen density. It is also shown by statistical data that pavement mixtures cannot be recompacted with the old compactor spring following the

standard California procedure to produce density and Hveem stability values comparable to field results.

14. Aggregate degradation studies show that maximum aggregate degradation in service has occurred at, or before, three years. The gradations at three and five years are not significantly different and they approach Fuller's maximum density gradation. Laboratory compacted samples were degraded to nearly the same gradation at all compaction pressures when the old spring was used with impact occurring above 450 psi, which shows that low pressures of 50 to 100 psi and without impact produce degradation of the same magnitude as obtained when high pressures with impact are used. The laboratory-compacted samples also approached Fuller's gradation and, on the average, the gradations resulting when samples were compacted near the design compaction pressure assigned to each mixture in this study usually differ from the 1957 or 1959 field gradations by one to two percent for any fraction. The largest deviations are in the two coarse aggregate fractions above the No. 4 sieve size.

15. The method of calibrating the kneading compactor, as outlined in Appendix C, is satisfactory. However, the area under the loading curve is not an accurate means of comparing foot pressures for different conditions of operation if the types of load imposed are not similar. Calibration on the basis of peak foot pressure is satisfactory only when similar load-cycle patterns are followed for all conditions. The differences in the load cycles produced with the two springs referred to throughout this study do not permit a simple means of comparison to be made with accurate results primarily because of the high impact with the old spring.

SUGGESTIONS FOR FURTHER RESEARCH

Numerous topics for continued research, in the area of study which encompasses this investigation, were presented as the work progressed. The extreme versatility of the kneading compactor offers many opportunities to interested researchers to answer many of the questions which are so prominent concerning compaction of soils and bituminous mixtures. The Stabilometer also offers excellent opportunity to evaluate the strength of a material and predict the performance of this material under highly variable conditions of service. The ease of handling and the ready adaptability of the Stabilometer to variable test conditions make it an instrument which extends several projects to engineering personnel who are interested in better establishing current design techniques.

This investigation has pointed out that much time and unnecessary testing can be saved by resorting to the use of a sensitive mixture for studies of the type discussed here. Some check tests with a less-sensitive mixture may be desired, but this study has shown that the Gradation A mixture consisting of all crushed limestone aggregate and 7.0 percent asphalt was very sensitive to any variables studied. It is suggested that these facts be considered when undertaking any further research outlined below.

Several of the ideas arrived at, while continuing this study, have been incorporated into this investigation. Other, more time-consuming projects are outlined here. Several of these are projects requiring

only a series of test results that can be obtained by skilled technicians. Other projects require continued study of the basic fundamentals involved before satisfactory procedures can be adopted for laboratory testing. A brief outline of suggested projects follows:

1. The conclusions from this study are very limited since three pavements have been studied which are subjected to quite similar conditions. It would be most informative to extend work of this type to apply to more highway conditions and a greater number of bituminous mixtures. In order to establish reasonable design criteria of Hveem stability and voids it would also be necessary to obtain data for pavements showing varying degrees of performance. The general procedure and testing pattern has been presented in this study but the results cannot be related to all bituminous pavements in Indiana.

2. As discussed under the CONCLUSIONS section of this report it is most essential to simulate the field condition of a pavement when compacting a laboratory sample. It is believed that only a kneading-type action during compaction will reproduce the aggregate degradation and particle orientation obtained in service. Furthermore, this study has shown that the Triaxial Institute kneading compactor does not reproduce the pavement condition in Indiana. It is suggested that the kneading-type action of the gyratory compactor be studied with the intention of developing a compaction procedure which will reproduce field density and stability conditions but, preliminary to this, it is necessary that gyratory compaction produce aggregate degradation and particle orientation as they occur in service.

3. Further applications of the kneading compactor to the design

of Indiana bituminous mixtures should certainly consist of varying the asphalt content to obtain design data for typical mixtures when using design compaction pressures presented by this study or by similar means. It would also be most helpful to obtain data for typical Indiana bituminous mixtures to determine the effects of varying several steps of the standard Hveem test procedure such as the Stabilometer test-speed and the sample curing time. Data to correlate the Hveem stability values of built-up specimens using a varying number of layers with specimens formed in the regular manner would also be most helpful before continuing any further research with the Hveem method where built-up specimens would be used extensively.

4. It is suggested that more work be undertaken to study the effect of varying the compaction time when a standard pressure is used. Preferably this standard pressure should be 500 psi since the kneading compactor is built to operate efficiently in this pressure range. A design compaction procedure for several mixtures could be obtained in this manner with compaction time being the only variable. In an effort to better simulate the field condition it is further suggested that specimens be molded in more than one layer, preferably, using layer thicknesses common to construction methods. This implies that a reasonable beginning point for Indiana conditions would be to compact laboratory surface specimens in three layers and binder specimens in two layers. Several combinations of variable compactive efforts for each layer would reasonably present a trial and error means of obtaining a compaction procedure to produce a specimen which would truly simulate field conditions, but the number of trials involved may indicate that this method would not be practical.

5. There is a possibility that valuable information can be obtained by establishing a method of recompacting pavement mixtures in the laboratory. The compaction techniques described above under 4) could be applied here. It is believed that no additional degradation of appreciable magnitude results from recompacting a mixture if the aggregate has been degraded in service to nearly a maximum density. Thus, density and stability results obtained from a recompacted specimen using the pavement mixture should be comparable to density and stability results for a laboratory-prepared sample using the original construction mixture. A correlation of the Hveem method with pavement performance based on Hveem stability, using pavement samples and recompacted samples, might be possible. This latter idea might be valuable to establish a correlation between laboratory-compacted and pavement Hveem stability values when the standard compaction procedure is used in the laboratory. Continued research is needed to disclose the true potential of these ideas.

6. The versatility of the kneading compactor is mentioned several times throughout this report. The machine can easily be used as a repeated-loading device with some minor modifications of the mechanism. Loading pressures for a 4-in. diameter specimen can readily be varied to a maximum of at least 200 psi according to the kneading compactor calibration resulting from this study. The compactor table can be adjusted to apply concentric loading and the detachable compactor foot can be easily replaced with several types of loading heads. Specimens can be compacted up to 12-in. in height. The true value of repeated-loading testing as related to the stability problems with bituminous mixtures in Indiana is still a somewhat questioned topic but the kneading compactor can be used as a loading device as outlined here.

LIST OF REFERENCES

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1. Abson, G., "Method and Apparatus for the Recovery of Asphalt," Proceedings, American Society for Testing Materials, Vol. 33, Part II, 1933.

2. Bennett, C. A. and Franklin, N. L., Statistical Analysis in Chemistry and the Chemical Industry, John Wiley and Sons, Inc., New York, 1954.

3. Bone, A. J., Crump, L. W. and Roggeveen, V. J., "Control of Reflection Cracking in Bituminous Resurfacing Over Old Cement-Concrete Pavements," Proceedings, Highway Research Board, Vol. 33, 1954.

4. Bulletin 93, Highway Research Board, Soil Density and Stability, 1954.

5. Bulletin 105, Highway Research Board, Bituminous Paving Mixtures - Fundamentals for Design, 1955.

6. Chen, C. C., "A Laboratory Study of the Stability of Bituminous Mixtures," A Thesis submitted to Purdue University for the degree of Master of Science in Civil Engineering, 1950.

7. Dennis, J. H., "The Effect of Density on the Repeated-Load Strength Properties of Bituminous Concrete," A Thesis submitted to Purdue University for the degree of Master of Science in Civil Engineering, 1959.

8. Dillard, J. H., "Comparison of Density of Marshall Specimens and Pavement Cores," Proceedings, The Association of Asphalt Paving Technologists, Vol. 24, 1955.

9. Endersby, V. A., "The History and Theory of Triaxial Testing, and the Preparation of Realistic Test Specimens - A Report of the Triaxial Institute," Triaxial Testing of Soils and Bituminous Mixtures, American Society for Testing Materials, Special Technical Publication No. 106, 1950.

10. Endersby, V. A. and Vallerga, B. A., "Laboratory Compaction Methods and Their Effects on Mechanical Stability Tests for Asphaltic Pavements," Proceedings, The Association of Asphalt Paving Technologists, Vol. 21, 1952.

11. Foster, C. R., "Reduction in Soil Strength with Increase in Density," Proceedings, American Society of Civil Engineers, Separate No. 228, 1953.

12. Goetz, W. H., "Flexible Pavement Research," Proceedings, Thirty-Sixth Annual Purdue Road School, 1950.

13. Goetz, W. H., "Comparison of Triaxial and Marshall Test Results," Proceedings, The Association of Asphalt Paving Technologists, Vol. 20, 1951.

14. Goetz, W. H., "Flexible Pavement Test Sections for Studying Pavement Design," Proceedings, Thirty-Seventh Annual Purdue Road School, 1952.

15. Goetz, W. H. and McLaughlin, J. H., "Performance Data for Some Bituminous Concrete Overlays in Indiana, " Proceedings, Forty-Third Annual Purdue Road School, 1957.

16. Goetz, W. H., McLaughlin, J. H. and Wood, L. E., "Load-Deformation Characteristics of Bituminous Mixtures Under Various Conditions of Loading," Proceedings, The Association of Asphalt Paving Technologists, Vol. 26, 1957.

17. Griffith, J. M. and Kallas, B. F., "Influence of Fine Aggregates on Asphaltic Concrete Paving Mixtures," Proceedings, Highway Research Board, Vol. 37, 1958.

18. Hannan, R. A., "Application of the Hveem Stabilometer to the Testing of Open-Graded Bituminous Mixtures," A Thesis submitted to Purdue University for the degree of Master of Science in Civil Engineering, 1959.

19. Hennes, R. G., "The Quest for Rational Pavement Design," Proceedings, The Association of Asphalt Paving Technologists, Vol. 25, 1956.

20. Herrin, M. and Goetz, W. H., "Effect of Aggregate Shape on Stability of Bituminous Mixes," Proceedings, Highway Research Board, Vol. 33, 1954.

21. Hveem, F. N., "Gradation of Mineral Aggregates for Dense Graded Bituminous Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Vol. 2, 1940.

22. Hveem, F. N. and Carmany, R. M., "The Factors Underlying the Rational Design of Pavements," Proceedings, Highway Research Board, Vol. 28, 1948.

23. Hveem, F. N., Application of the Triaxial Test to Bituminous Mixtures - Hveem Stabilometer Method, California Division of Highways, 1949.

24. Hveem, F. N. and Davis, H. E., "Some Concepts Concerning Triaxial Compression Testing of Asphaltic Paving Mixtures and Subgrade Materials, Triaxial Testing of Soils and Bituminous Mixtures, American Society for Testing Materials, Special Technical Publication No. 106, 1950.

25. Hveem, F. N. and Vallerga, B. A., "Density Versus Stability," Proceedings, The Association of Asphalt Paving Technologists, Vol. 21, 1952.

26. Lottman, R. P. and Goetz, W. H., Effect of Crushed-Gravel Fine Aggregate on the Strength of Asphaltic Surfacing Mixtures, National Sand and Gravel Association, Washington, D. C., 1956.

27. Marshall, B. G., Marshall Stability Method for the Design and Control of Asphalt Paving Mixtures, Published by the author, 1946.

28. McCarty, L. E., "Correlation Between Stability and Certain Physical Properties of Bituminous Materials," Proceedings, Highway Research Board, Vol. 33, 1954.

29. McLeod, N. W., "Application of Triaxial Testing to the Design of Bituminous Pavements," Triaxial Testing of Soils and Bituminous Mixtures, American Society for Testing Materials, Special Technical Publication No. 106, 1950.

30. McRae, J. L., "Compaction of Bituminous Concrete," Proceedings, The Association of Asphalt Paving Technologists, Vol. 26, 1957.

31. McRae, J. L. and McDaniel, A. R., "Progress Report on the Corps of Engineers' Kneading Compactor for Bituminous Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Vol. 27, 1958.

32. Monismith, C. L. and Vallerga, B. A., "Relationship Between Density and Stability of Asphaltic Paving Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Vol. 25, 1956.

33. Monismith, C. L., Personal Communication, 1960.

34. Neppe, S. L., "Mechanical Stability of Bituminous Mixtures: A Summary of the Literature," Proceedings, The Association of Asphalt Paving Technologists, Vol. 22, 1953.

35. Nevitt, H. G., "Compaction Fundamentals," Proceedings, The Association of Asphalt Paving Technologists, Vol. 26, 1957.

36. Oppenlander, J. C., "Triaxial Testing of Bituminous Mixtures at High Confining Pressures," A Thesis submitted to Purdue University for the degree of Master of Science in Civil Engineering, 1957.

37. Oppenlander, J. C. and Goetz, W. H., "Triaxial Testing of Open-Type Bituminous Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Vol. 27, 1958.

38. Oppenlander, J. C. and Goetz, W. H., "Triaxial Testing of Bituminous Mixtures at High Confining Pressures," Proceedings, Highway Research Board, Vol. 37, 1958.

39. Philippi, O. A., "Compaction of Bituminous Concrete," Proceedings, The Association of Asphalt Paving Technologists, Vol. 26, 1957.

40. Rice, J. M., "Maximum Specific Gravity of Bituminous Mixtures by Vacuum Saturation Procedure," Preprint, American Society for Testing Materials, 1956.

41. Roberts, S. E., "Cracks in Asphalt Resurfacing Affected by Cracks in Rigid Bases," Proceedings, Highway Research Board, Vol. 33, 1954.

42. Schaub, J. H., "Strength and Volume Change Characteristics of a Bituminous Mixture Under Triaxial Testing," A Thesis submitted to Purdue University for the degree of Doctor of Philosophy, 1960.

43. Smith, V. R., "Application of the Triaxial Test to Bituminous Mixtures California Research Corporation Method," Triaxial Testing of Soils and Bituminous Mixtures, American Society for Testing Materials, Special Technical Publication No. 106, 1950.

44. Stanton, T. E. and Hveem, F. N., "Role of the Laboratory in the Preliminary Investigation and Control of Materials for Low Cost Bituminous Pavements," Proceedings, Highway Research Board, Vol. 14, Part II, 1934.

45. State Highway Department of Indiana, Standard Specifications for Road and Bridge Construction and Maintenance, 1957.

46. State of California, Department of Public Works, Division of Highways, Standard Specifications, 1954.

47. State of California, Department of Public Works, Division of Highways, Materials Manual of Testing and Control Procedures, Vols. I and II, 1956.

48. Stevens, D. E., "Fundamentals of Stability Testing of Asphalt Mixes," Proceedings, The Association of Asphalt Paving Technologists, Vol. 22, 1953.

49. The Asphalt Institute, Mix Design Methods for Hot-Mix Asphalt Paving, Manual Series No. 2, College Park, Maryland, 1956.

50. Vallerga, B. A., "Recent Laboratory Compaction Studies of Bituminous Paving Mixtures," Proceedings, The Association of Asphalt Paving Technologists, Vol. 20, 1951.

51. Vallerga, B. A., "Getting Better Pavement Performance," Proceedings, Northwest Conference on Road Building, 1954.

52. Vallerga, B. A., "On Asphalt Pavement Performance," Proceedings, The Association of Asphalt Paving Technologists, Vol. 24, 1955.

53. Vallerga, B. A., "The Triaxial Institute and the Stabilometer," Proceedings, Highway Research Board, Vol. 34, 1955.

54. Wood, L. E. and Goetz, W. H., "The Strength of Bituminous Mixtures and Their Behavior Under Repeated Loads," Proceedings, Highway Research Board, Vol. 35, 1956.

55. Zube, E., "Correlation Between Pavement Design and Performance," Proceedings, Northwest Conference on Road Building, 1954.

56. Williams, F. M. and Kimble, F. W., "Effects of Mixture Coarseness, Mix Temperatures, and Course Thicknesses on Field Density of Bituminous Concrete," Proceedings, The Association of Asphalt Paving Technologists, Vol. 23, 1954.

APPENDIX

APPENDIX A

DESCRIPTION AND OPERATION OF THE HVEEM STABILOMETER

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DESCRIPTION AND OPERATION OF THE HVEEM STABILOMETER

It is not intended to present a detailed discussion of the operation of the Stabilometer since readily available literature, such as the California Materials Manual (47), covers this subject thoroughly. However, some of the fundamentals involved are presented with a brief description of the testing procedure.

The Stabilometer is compact, sturdy, and easily operated, and when correctly used it yields reproducible results which are susceptible to physical interpretation in terms of shear strength of the material. In addition to meeting these requirements for an adequate test, it has been reported that the test results have a high degree of correlation with field performance (24).

The specimen size is 4 in. in diameter and $2\frac{1}{2}$ in. high, although provisions can be made by an adjustable stage to test specimens ranging from 2.2 in. to 3.0 in. in height. Charts are available to correct test results to strength values corresponding to $2\frac{1}{2}$ in. high specimens.

The Stabilometer measures primarily friction between the aggregate particles. Thus, test results are not greatly influenced by ordinary temperature changes or by the viscosity of the bituminous binder. This is particularly true since the test is conducted at 140°F, whereas a test at a lower than room temperature would result in some effect on the test result.

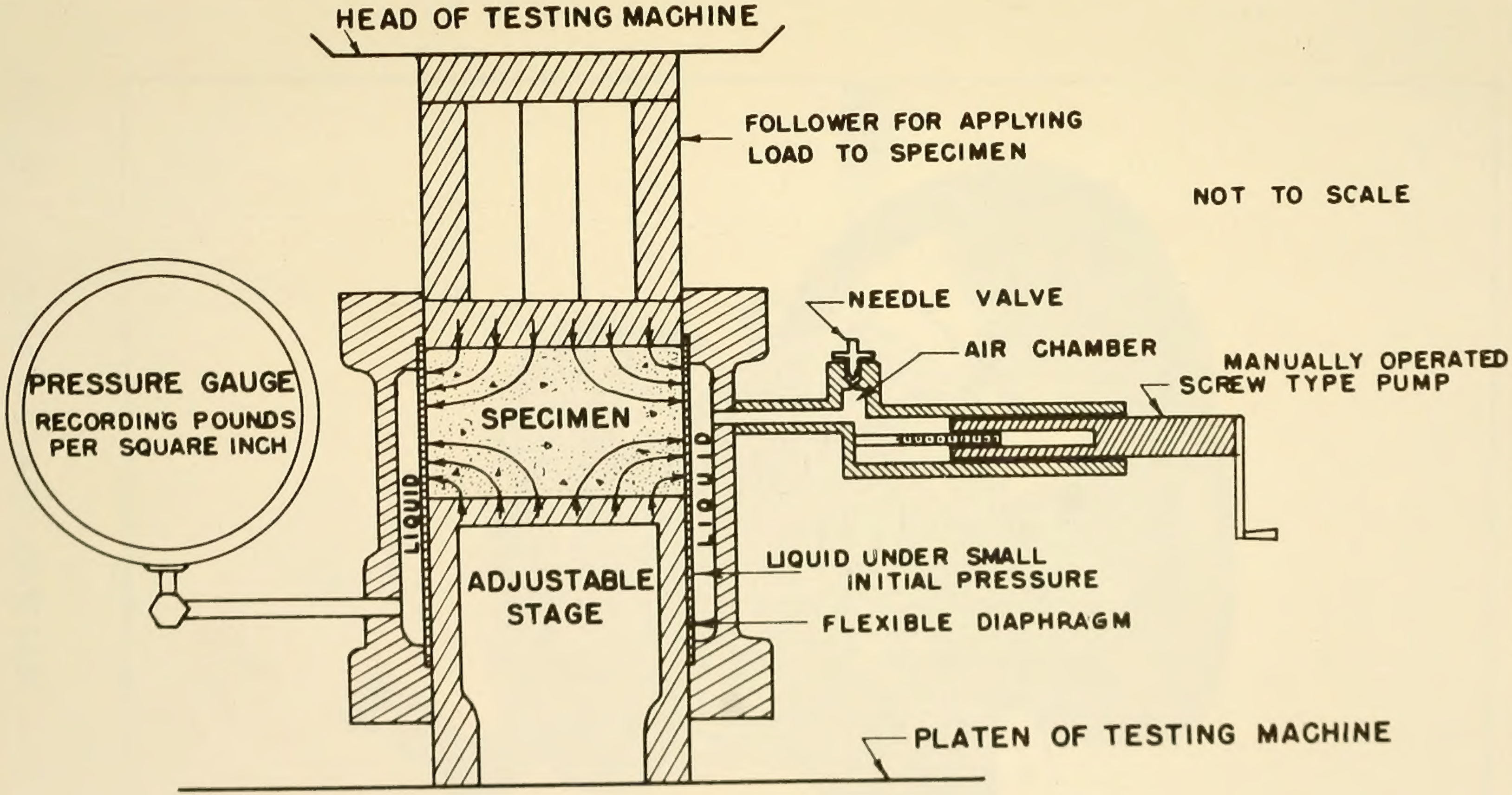
The artificial curing period of at least 16 hours at 140°F with

provision for air circulation is very essential since curing of the mixture will have a marked effect on the test results under a fixed rate of strain. Hveem and Davis (24) state that the artificial curing time is reasonable when compared to conditions on the road after placing.

For details of the test procedure the reader is referred to the California Manual (47), Hannan (18), Highway Research Board Bulletin 105 (5), Asphalt Institute Manual on Hot-Mix Asphalt Paving (49) and Zube (55). It suffices here to state that the Hveem Stabilometer test is a type of "closed system" triaxial compression test. The vertical load is applied at a constant strain rate (0.05 in. per minute) while pressure is allowed to build up in the liquid cell which encircles and confines the specimen laterally. The Stabilometer value is obtained from the transmitted horizontal pressure of 400 psi vertical load and is expressed in a scale which ranges from 0 to 100. This stability value is obtained by use of a hyperbolic equation which has been presented as a closure under the section heading, REVIEW OF LITERATURE.

In a pure fluid of zero stability, the transmitted pressure must equal the vertical load. For complete stability the transmitted pressure must be zero. California has set minimum laboratory compacted bituminous concrete stability limits of 30 and 35 for medium and heavy traffic, respectively. California regards stability values of less than 25 to indicate an undesirable mixture regardless of the other properties of the mixture.

Figure 30 demonstrates the various components of the Stabilometer and Figure 31 pictures the assembled Stabilometer ready for testing.



NOTE: SPECIMEN GIVEN LATERAL SUPPORT BY FLEXIBLE SIDE WALL WHICH TRANSMITS HORIZONTAL PRESSURE TO LIQUID. MAGNITUDE OF PRESSURE MAY BE READ ON GAUGE

SOURCE: REFERENCE 47

FIG. 30 DIAGRAMATIC SKETCH OF THE HVEEM STABILOMETER

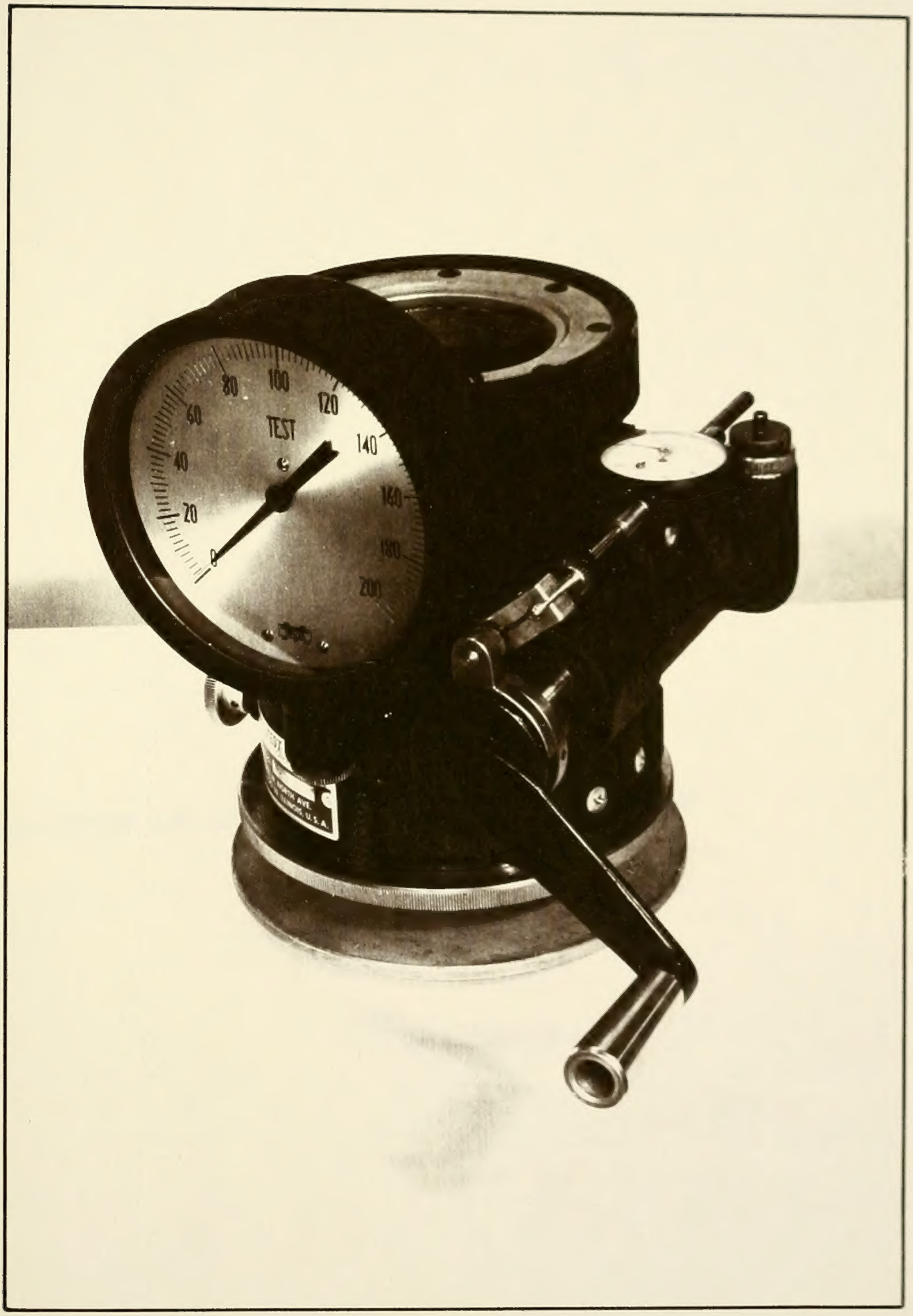


FIG. 31 HVEEM STABILOMETER

APPENDIX B

DESCRIPTION AND OPERATION OF THE KNEADING COMPACTOR

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DESCRIPTION AND OPERATION OF THE KNEADING COMPACTOR

The kneading compactor used in this investigation was the most recent model manufactured by the August Manufacturing Company in Oakland, California. The machine is pictured in Figure 32.

Over the past ten years the kneading compactor has undergone numerous revisions in an effort to make the machine more compact while still having a semi-automatic machine in which the personal error is greatly reduced. Some automatic features, such as the feeder trough, have been eliminated in an effort to reduce the cost of the unit.

Another modification has been to install a coil spring in the tamper arm. The spring helps to decrease the rate of load application in order to eliminate inertia effects of the hydraulic portion of the control system.

The compactor applies a series of individual tamps or fleeting pressures to the specimen through a tamping foot shaped like a segment of a circle. The table supporting the forming mold rotates one-sixth of a turn between tamps so that the tamper strikes a different area each time. The action is such that in any one tamp the pressure is gradually built up, in order to avoid impact, and then allowed to dwell on the specimen for a fraction of a second before being gradually released. The purpose of this dwelling period is to overcome effects of viscosity of the bitumen. The compaction rate is 30 tamps or cycles per minute. A typical trace of the load cycle is shown in Figure 33

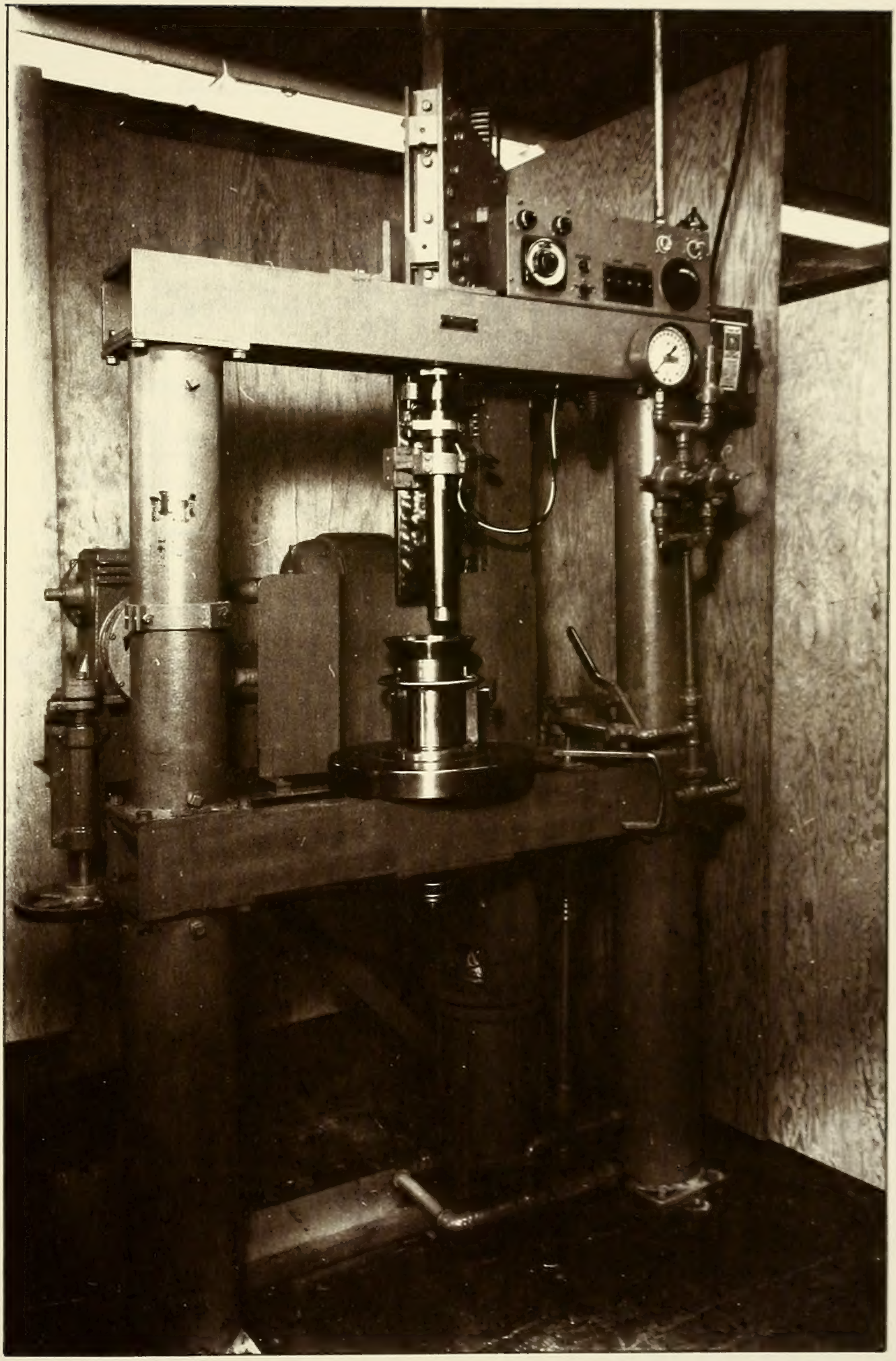
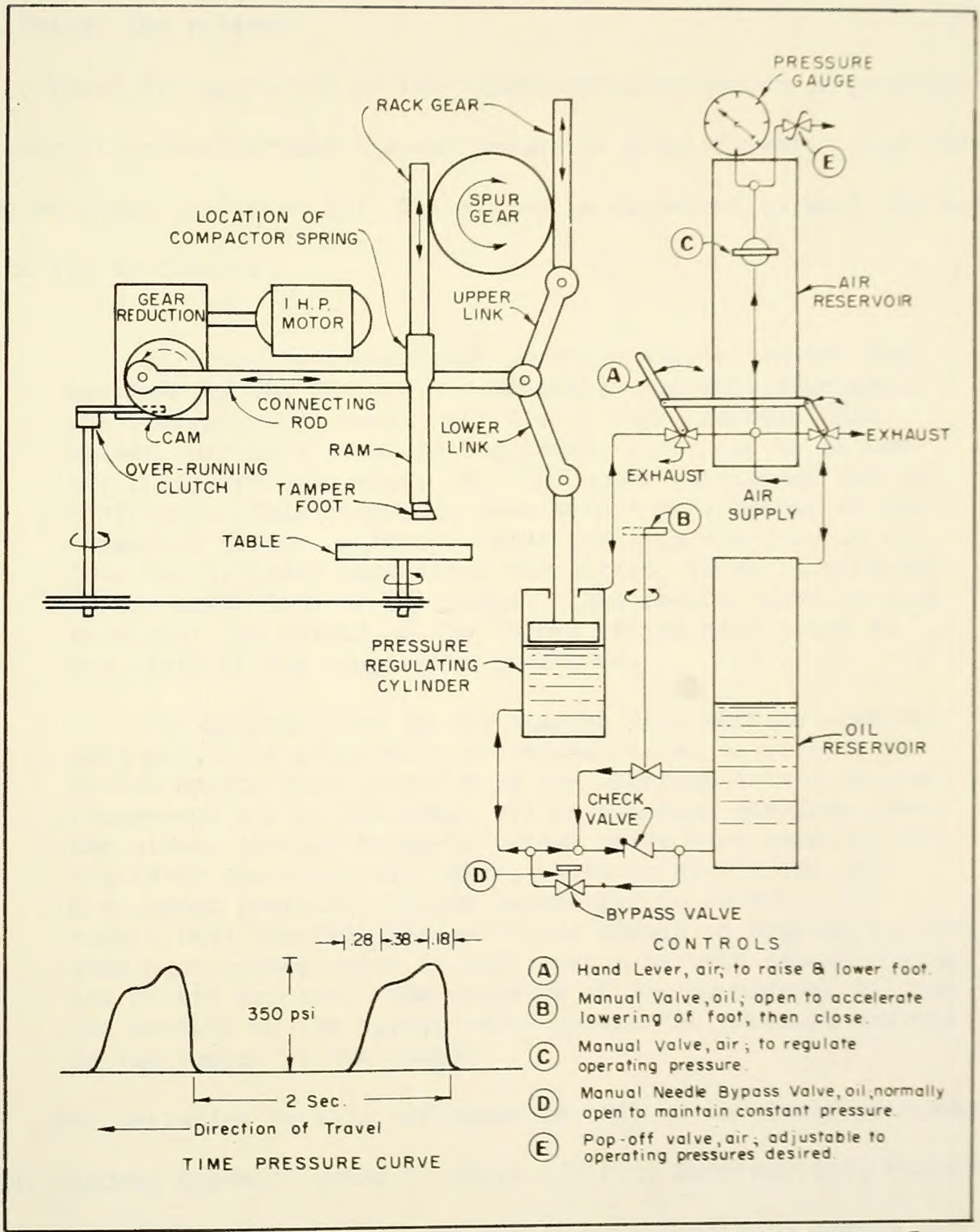


FIG. 32 MECHANICAL KNEADING COMPACTOR



SOURCE: REFERENCE 47

FIG. 33 SCHEMATIC DIAGRAM OF THE KNEADING COMPACTOR

and it will be seen that the pressure-versus-time curves consist of three distinct parts: first, the application; second, the dwell; and third, the release.

Power for operation of the toggle-press mechanism is provided by an electric motor through a speed reduction gear, flywheel, and connecting road as shown in Figure 33. The system is explained by Seed and Monismith (4) as follows:

> In order to control the pressure exerted on the test specimen by the tamper, a combination hydraulic-pneumatic control system is used. Air from a high-pressure line passes through a pressure regulator, which can be set at any predetermined value, into the upper portion of the oil reservoir. This reservoir also serves as a member of the compactor frame. A feeder valve controls the flow of oil into the cylinder containing the piston, which is attached to the lower link of the press. This feeder valve is used to adjust the height of the tamper in the mold prior to the start of the compacting procedure.
>
> The maximum load on the tamping foot remains constant throughout the compaction procedure since, as soon as the piston exerts more pressure on the oil than exists in the compressed air in the tank, oil is squeezed out from under the piston through a one-way check valve back into the oil reservoir and a pop-off valve, which is set at the predetermined pressure, allows excess air to escape. To ensure that the full pressure will always be applied to the sample, a bypass valve is kept open a certain amount during the entire process. The pressure of the compressed air and the setting of the bypass valve govern the pressure exerted by the tamper on the sample.

For operation details reference is made to the California Manual (47), Highway Research Board Bulletin 105 (5), Endersby (9), Vallerga (50), Seed and Monismith (4), and the Asphalt Institute Manual on Hot-Mix Asphalt Paving (49).

APPENDIX C

CALIBRATION OF THE KNEADING COMPACTOR

APPENDIX C

CALIBRATION OF THE KNEADING COMPACTOR

Before the compactor can be used it is first necessary to measure the load exerted by the tamper foot at different air pressures and settings of the bypass valve. Calibration also provides pressure-time curves showing the load-application cycle which consists of the application, the dwell, and the release. If there are any mechanical and/or hydropneumatic faults in the system, the calibration will detect these irregularities.

Reference is made to Appendix B for a picture of the compactor (Figure 32) and a diagramatic sketch of the control system (Figure 33). A discussion of the procedure and method of analyzing the results follows.

California Methods

The calibration system common now at the University of California is described at length by Vallerga (50). Other sources of information are Seed and Monismith (4) and Endersby (9), and the California Materials Manual (47) for the California Highway Department procedure. The methods consist of placing four electrical strain gages on a load-carrying member of the compactor. The equipment used for this operation consists of a voltage regulator, an amplifying unit, a rectifier, an oscillograph, and a recording unit.

Normally, an aluminum tubular compression dynamometer is inserted immediately above the tamper foot. Four SR-4 A-200 strain gages are

cemented 90 degrees apart on the outside surface of the tube, two diametrically opposite, with their axes parallel to the axis of the tube, and two, diametrically opposite, with their axes normal to the axis of the tube. An aluminum tube is used rather than steel because of its lower modulus or elasticity giving correspondingly higher strains.

This calibration method has found acceptance because foot pressure and load-application curves can readily be obtained while a soil or bituminous sample is being compacted in the usual manner. Monismith (33) states that a bypass valve setting of $1\frac{1}{2}$ turns has generally been found to produce the desired type of trace but, for any given air pressure, the peak foot pressure will increase as the bypass valve opening is increased.

Equipment and Procedures

The procedure used to calibrate the machine in this investigation differs considerably from the method briefly described above. The method used for this study involves the use of less equipment, but does not have the versatility of the University of California method. Typical soil or bituminous concrete samples cannot be compacted by this procedure, which is an undesirable feature. Another disadvantage of the system is that only two strain gages were used and difficulty was encountered in obtaining the same reading when rotating the pedestal through a horizontal plane. This effect was produced as a result of the tamper foot being slightly non-level. This was overcome to some extent by placing a self-leveling cap on the pedestal, but the discrepancies obtained still warranted another step.

The problem was overcome, to a point of satisfaction, by taking

four sets of readings and averaging the results. The pedestal was placed in position and a set of readings was taken after which the cap of the pedestal was turned 180 degrees and another set of readings was recorded. The pedestal was then turned 180 degrees and the above two sets of readings were obtained for that position. Experience indicated that the average results obtained in this manner were highly reliable.

It is believed that the procedure as outlined is satisfactory for obtaining peak foot pressures for the purpose of obtaining a calibration curve for the compactor. Monismith (33) states that the traces obtained using this method are satisfactory.

The calibration system, as used, consisted of a compression dynamometer inserted immediately beneath the compactor foot. The dynamometer was made of aluminum to the dimensions shown in Figure 35 to form a pedestal. Two SR-4 A-7 strain gages were mounted on the surface of the load cell in diametrically opposite positions. The only additional equipment required was an amplifier and brush recorder for recording the pressure-time curve. Figure 34 pictures the equipment ready for calibration with proper control settings given in the legend, and Figure 35 is a diagram of this equipment showing proper electrical connections.

The first step of the procedure was to calibrate the dynamometer. This procedure is briefly outlined as follows:

1 Place a calibrated 4,000 pound capacity proving ring under the tamper foot and apply static loads at varying air pressures by allowing the air pressure to force the foot downward without engaging the electric motor.

2. Place the dynamometer under the tamper foot and apply static loads at the same air pressure settings as used when loading the proving

KEY

1. AMPLIFIER WITH CALIBRATION SET FOR 15 MM. DIVISIONS OFFSET. OPERATE WITH ATTENTUATOR FACTOR SETTING OF 5 WHEN CALIBRATING.
2. CAPACITOR WITH SETTING OF 0.01 MICROFARAD.
3. OSCILLOGRAPH RECORDER. OPERATE AT 25 MM. PER SECOND FOR DESIRED TRACE.

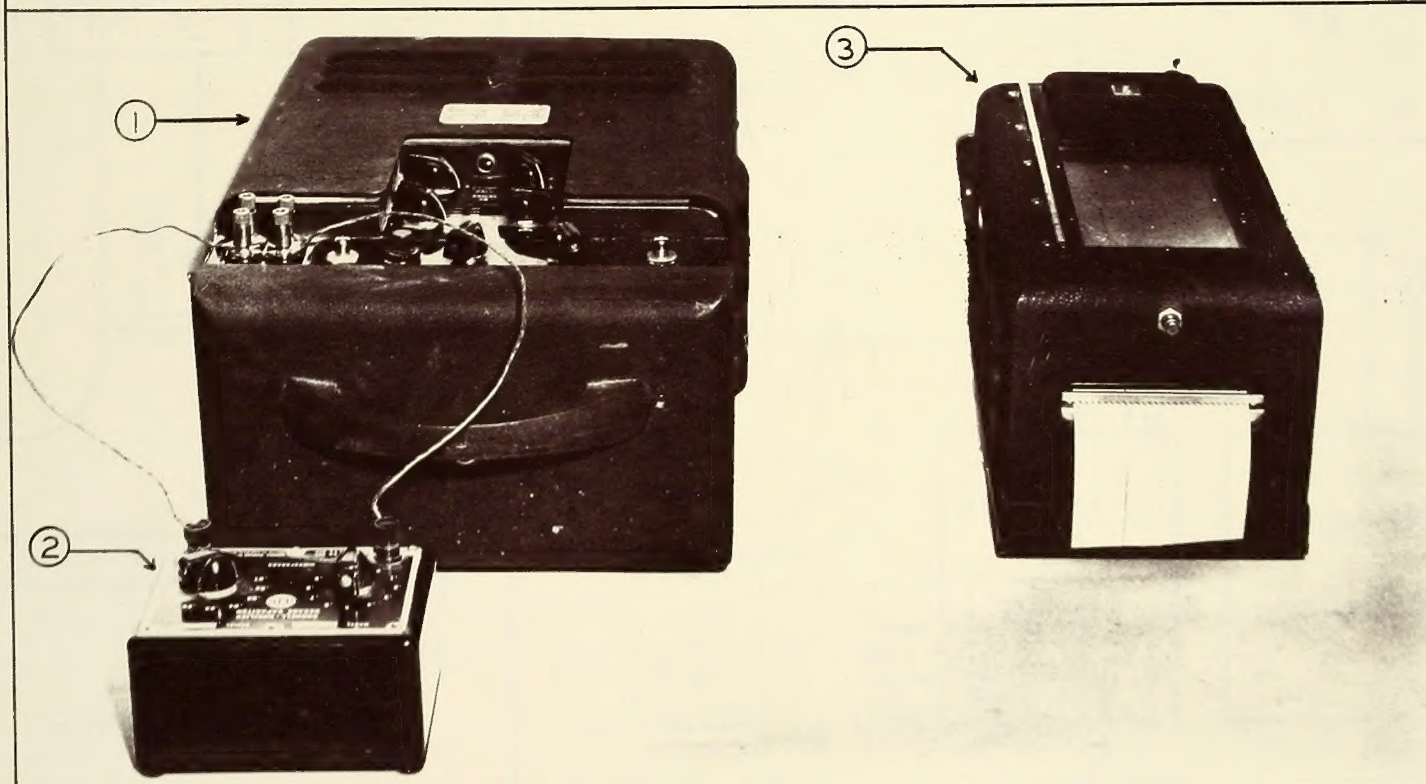


FIG. 34 INSTRUMENTS FOR CALIBRATING THE KNEADING COMPACTOR

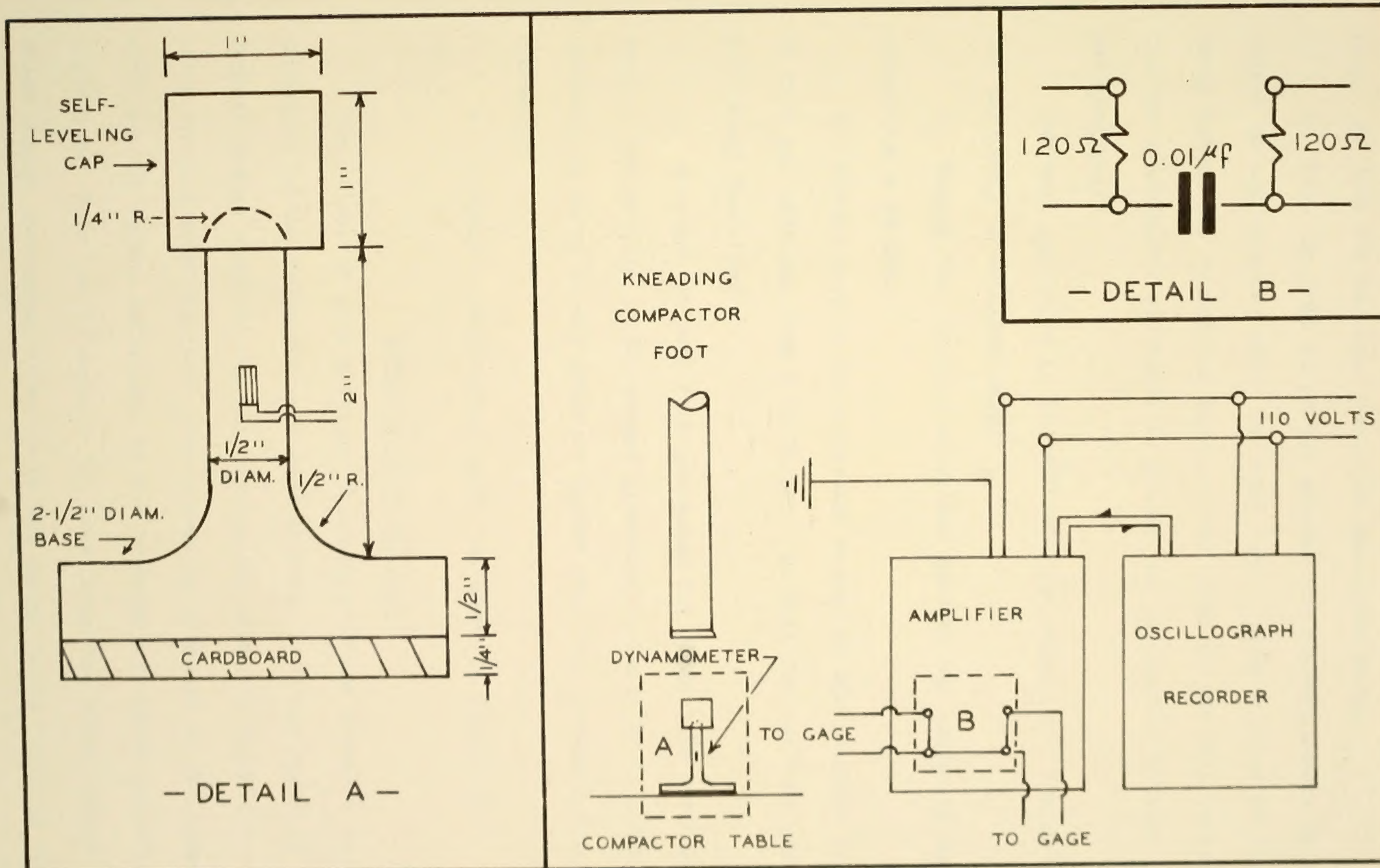


FIG. 35 DIAGRAM OF CALIBRATION EQUIPMENT

ring. Do this for the four positions described above.

3. Plot the load in pounds against the average number of divisions offset on the recorder paper for each load. Draw a curve to fit best these points and record the slope of the curve as the calibration factor for the dynamometer expressed in pounds per millimeter division on the recorder paper.

The next operation is to calibrate the compactor. The procedure for this step is outlined below:

1. Engage the electric motor and operate the compactor as if compacting a sample.

2. Obtain typical pressure-time traces at air pressure increments of five psi settings from 5 to 50 psi. Do this for the four positions previously described.

3. Average the peak foot pressures for these four settings and analyze the results in the manner presented in Table 22 to obtain a calibration curve of air pressure versus peak foot pressure for the compactor.

4. Prepare a calibration curve, as shown in Figure 38.

Interpretation of Data

The values of peak pressure obtained with the standard spring installed in the compactor were read directly from the recorded trace. Typical traces obtained for this spring are shown in Figure 36. In order to determine realistic peak pressures produced by the compactor with the old spring installed, it was necessary to establish a new criterion. This was to measure the area under the trace, since these areas should correspond directly to the energy input required to

Table 22

Typical Calibration Data for Kneading Compactor [a]

Gage Pressure, psi.	Peak Displacement, mm.					x Dynamometer Factor lbs./mm.	= Peak Load lbs.	÷ 3.2 Sq In Tamping Area	= Peak Foot Pressure, psi.
	Pos. 1	Pos. 2	Pos. 3	Pos. 4	Average				
5	3.6	3.1	4.2	5.7	4.15	85.8	356	3.2	111
10	6.0	5.2	6.5	7.8	6.4	85.8	549	3.2	172
15	8.9	8.1	9.4	10.2	9.15	85.8	785	3.2	245
20	11.8	11.0	12.2	13.0	12.0	85.8	1030	3.2	322
25	14.3	14.0	15.1	16.1	14.9	85.8	1278	3.2	399
30	17.1	16.8	17.8	19.0	17.7	85.8	1519	3.2	475
35	20.0	19.6	20.4	22.0	20.5	85.8	1759	3.2	550
40	23.0	22.5	23.3	24.0	23.2	85.8	1991	3.2	622
45	25.8	25.3	25.8	26.5	25.85	85.8	2218	3.2	693
50	28.0	28.2	28.0	28.6	28.2	85.8	2420	3.2	756

(a) Data are for standard spring.

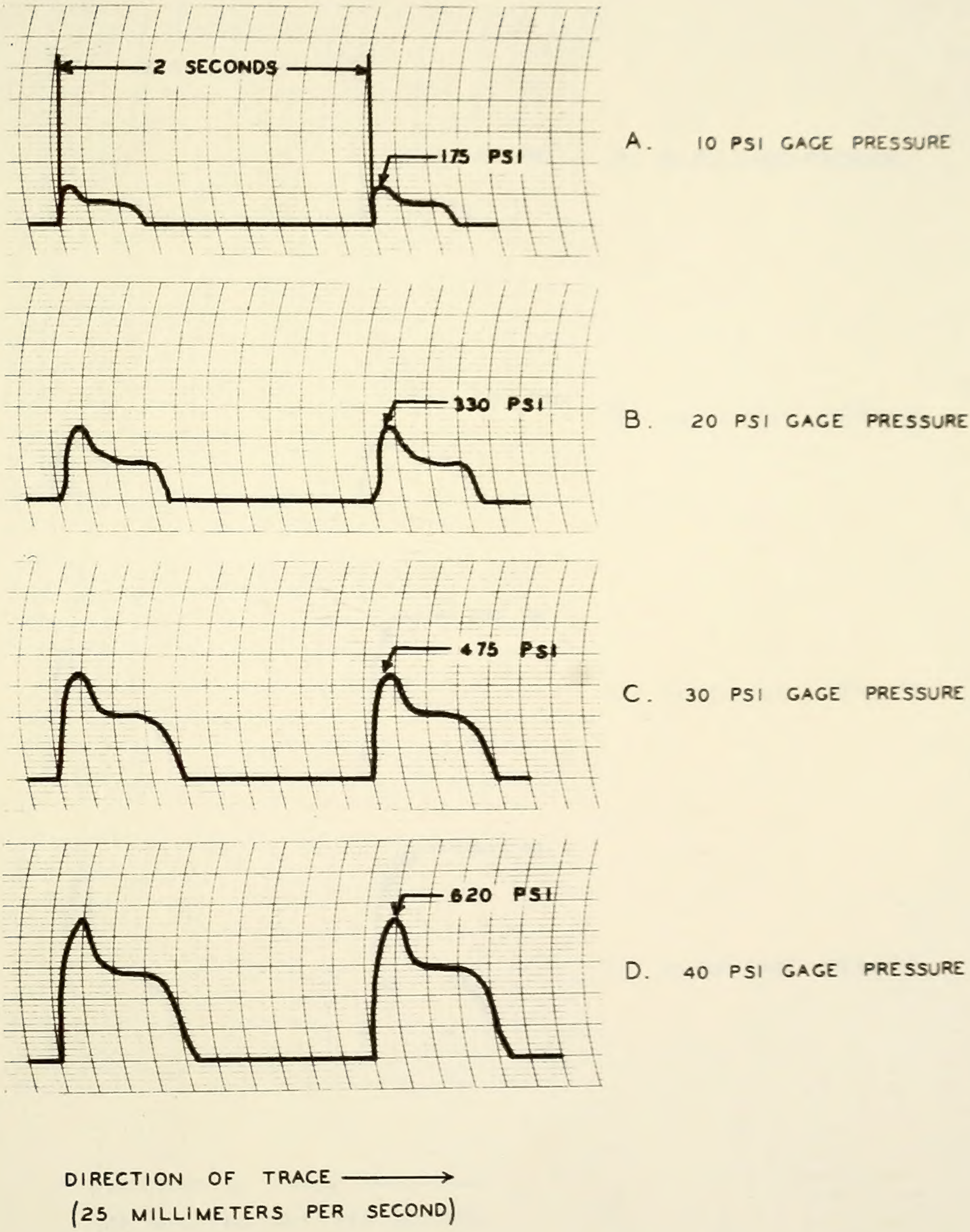


FIG. 36 TYPICAL TRACES WITH STANDARD SPRING - BYPASS VALVE OPEN 1-3/4 TURNS

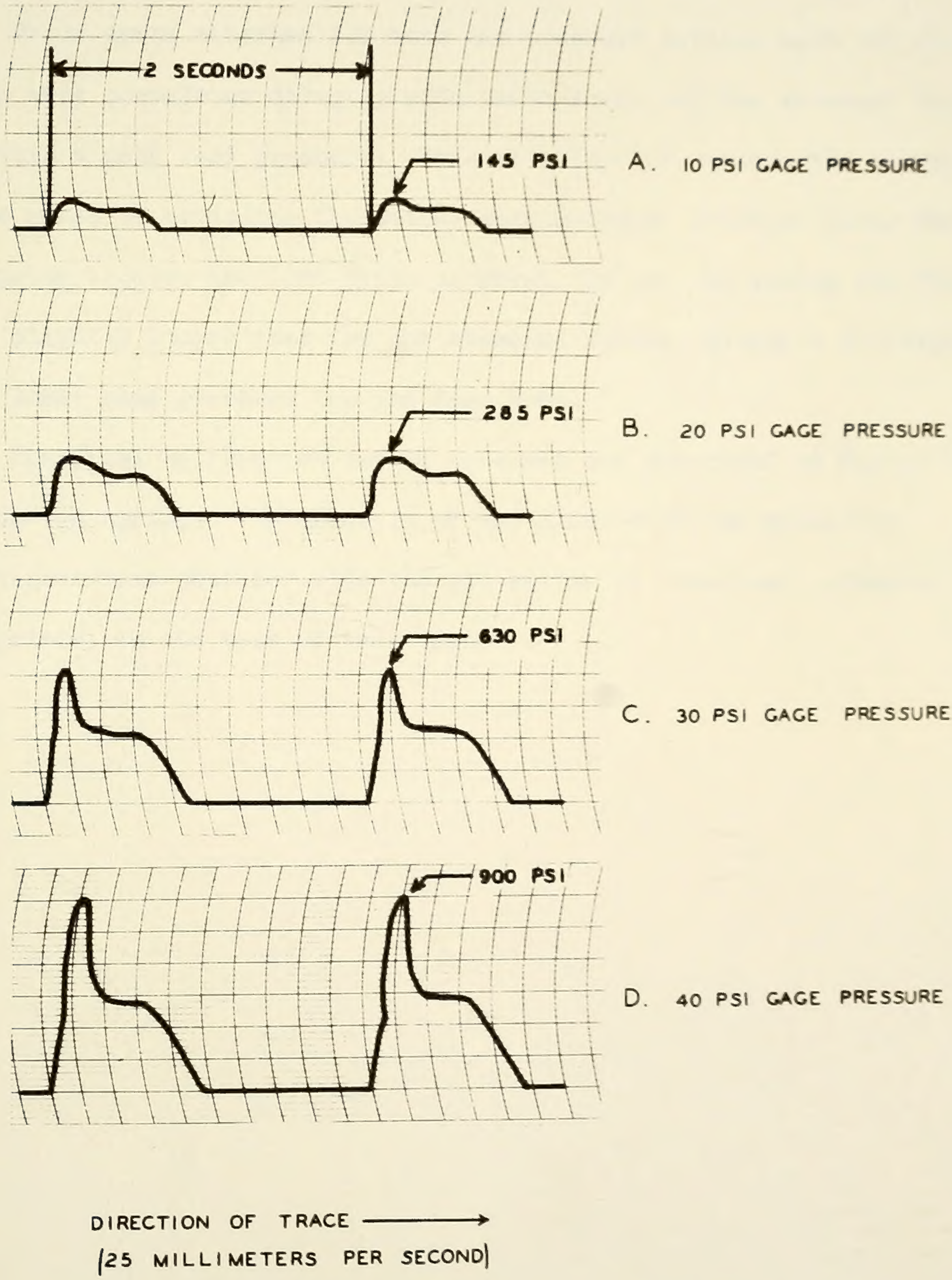


FIG. 37 TYPICAL TRACES WITH OLD SPRING - BYPASS VALVE OPEN 1-3/4 TURNS

compact samples over the range of the calibration. Typical traces for this spring are shown in Figure 37.

Trace areas obtained for each air pressure setting with the old spring were correlated directly with trace areas for the standard spring to obtain a peak foot pressure. This procedure is not exactly correct, in the sense of producing the actual peak pressure obtained under the old spring, because the load-cycle interval for the old spring was found to be slightly longer than for the standard spring, giving a correspondingly lower peak pressure for the same area.

The final calibration curves obtained are presented in Figure 38 for the two springs. Discussion of the effects of the excessive impact pressures obtained with the old spring is presented, whenever appropriate, in the text of this report.

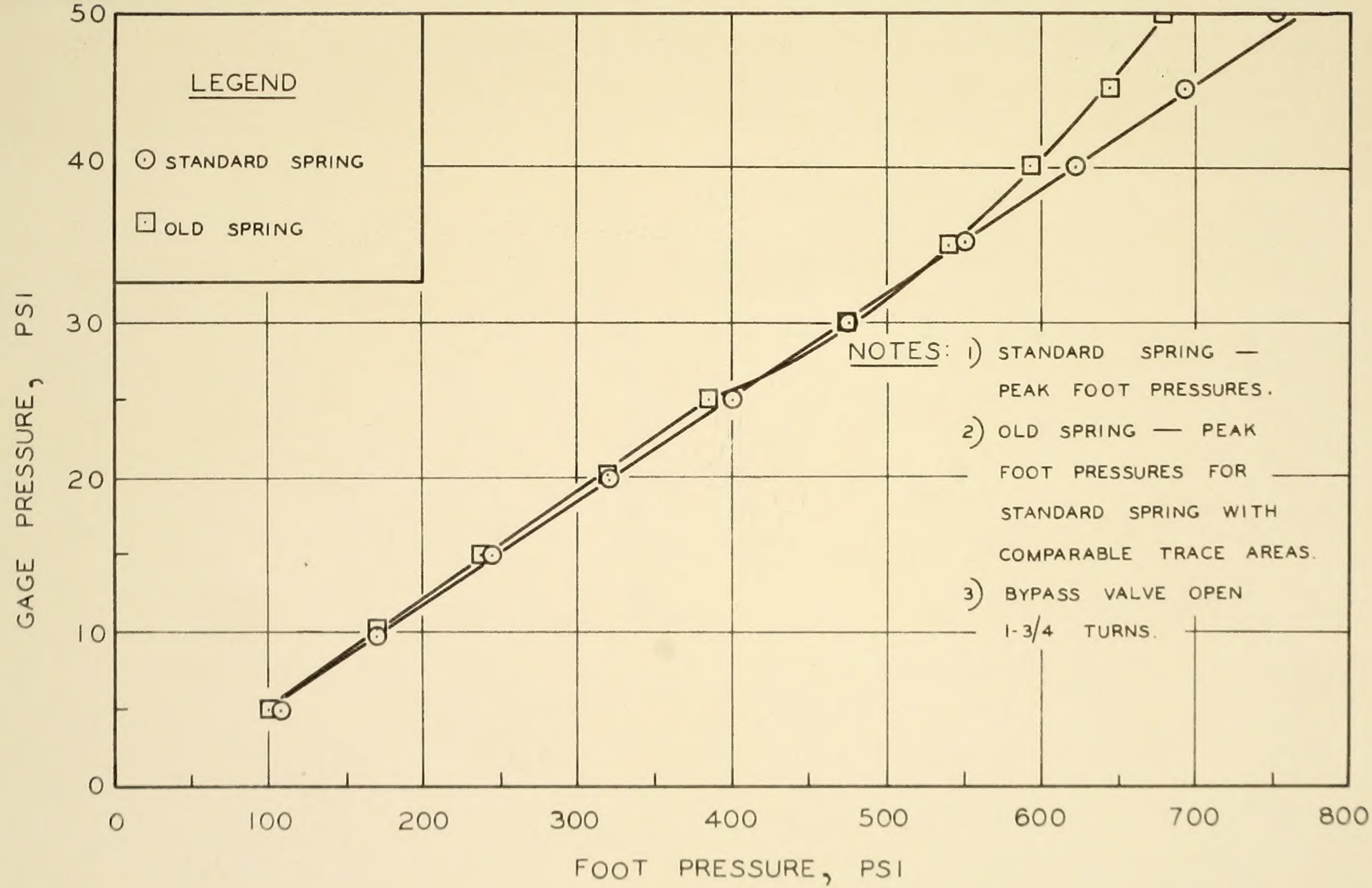


FIG. 38 CALIBRATION CURVES FOR KNEADING COMPACTOR

APPENDIX D

DATA - 1954, 1955, 1957

APPENDIX D

DATA - 1954, 1955, 1957

The test results presented in this section have been obtained by personnel of the Joint Highway Research Project Bituminous Laboratory over a three-year period. The complete test data have been summarized and tabulated in the form presented here.

This summary provides useful quantitative results relating the service performance of the three highways concerned over a period of three years. Test results are presented in Table 24 for the year of construction, one year after construction, and three years after construction. Supplementary results showing asphalt contents, mixture void contents, and aggregate gradations are included for three years after construction in Tables 25 and 26. An evaluation of these data is presented in the text of this report and these results have been combined with later test data to extend the performance study period to five years.

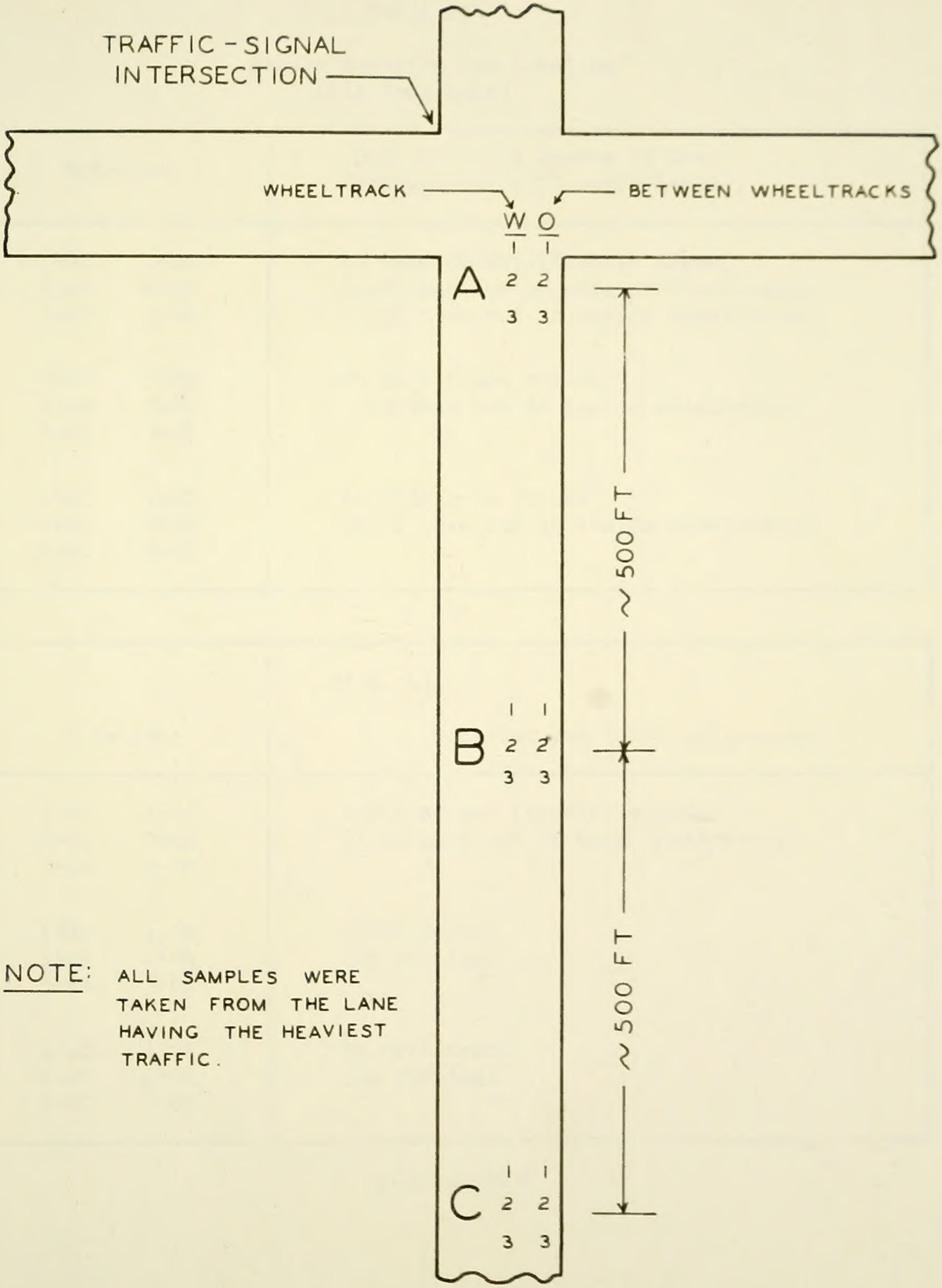


FIG. 39 TYPICAL SAMPLE LOCATION DIAGRAM
(ALL SAMPLINGS)

Table 23

Sample Notation and Location
(All Samplings)

	Notation		U.S. 20 - 5th Avenue in Gary Location and 1957 Performance
	1-WA	1-OA	At Taft Street (traffic signal)
A	2-WA	2-OA	(1/8 inch rut in outside wheeltrack)
	3-WA	3-OA	(1/16 inch rut in inside wheeltrack)
	1-WB	1-OB	At Taft Place Street
B	2-WB	2-OB	(1/8 inch rut in inside wheeltrack)
	3-WB	3-OB	
	1-WC	1-OC	At Ellsworth Street
C	2-WC	2-OC	(1/16 inch rut in inside wheeltrack)
	3-WC	3-OC	

	Notation		U.S. 41 Location and 1957 performance
	1-WA	1-OA	119th Street (traffic signal)
A	2-WA	2-OA	(1/16 inch rut in both wheeltracks)
	3-WA	3-OA	
	1-WB	1-OB	120th Street
B	2-WB	2-OB	(No rutting)
	3-WB	3-OB	
	1-WC	1-OC	At Restaurant
C	2-WC	2-OC	(No rutting)
	3-WC	3-OC	

(Continued)

Table 23 (continued)

Sample Notation and Location
(All Samplings)

Notation			U.S. 12 - Indianapolis Blvd. (a) Location
	1-WA	1-OA	At U.S. 41 Intersection (traffic signal)
A	2-WA	2-OA	
	3-WA	3-OA	
	1-WB	1-OB	At Tourist Court
B	2-WB	2-OB	
	3-WB	3-OB	
	1-WC	1-OC	North End of Rinso Soap Factory
C	2-WC	2-OC	
	3-WC	3-OC	

(a) To 1957, there was no excessive rutting in any of the sections sampled.

Table 24

Summary of Bulk Density and Marshall Test Results
(1954, 1955, 1957)

Specimen Number	Bulk Density (lb./cu.ft.) 1954	1955	1957	Marshall Stability (lbs.) 1954	1955	1957	Marshall Flow (1/100 in.) 1954	1955	1957
20-1-WA	142.3	152.7	152.5	533	2849	2424	21.3	24.8	13.4
2	141.6	151.9	152.8	840	2622	1931	21.1	23.9	29.9
3	142.9	152.2	152.2	878	2760	1523	19.3	14.2	28.0
Average	142.2	152.2	152.5	750	2744	1959	20.6	21.0	23.8
20-1-OA	144.1	151.3	154.3	803	2040	2822	17.9	18.9	19.1
2	145.4	150.5	154.3	1063	2102	2684	20.0	17.3	19.3
3	144.8	151.5	153.9	869	2070	3022	19.2	18.8	19.9
Average	144.8	151.1	154.2	912	2171	2843	19.0	18.3	19.4
20-1-WB	142.3	149.0	149.6	707	1562	1836	17.8	15.5	16.8
2	142.9	148.7	150.2	709	1683	1927	17.2	16.1	19.4
3	142.9	148.5	149.9	799	1371	2002	21.2	11.6	19.3
Average	142.7	148.7	149.9	738	1539	1922	18.7	14.4	18.5
20-1-OB	144.1	149.1	150.8	690	1891	1759	21.8	14.0	16.4
2	144.1	149.5	150.6	962	1600	1653	22.5	16.3	17.6
3	144.8	149.5	149.1	766	1781	1976	20.7	17.1	19.9
Average	144.3	149.4	150.2	806	1757	1796	21.7	15.8	18.0
20-1-WC	148.5	148.9	151.1	1131	1518	1754	25.9	13.0	26.0
2	147.3	148.2	151.1	1100	1488	1756	21.7	16.6	30.1
3	147.3	147.0	151.0	1224	1514	1576	22.2	15.6	26.1
Average	147.7	148.0	151.1	1152	1507	1695	23.3	15.1	27.4
20-1-OC	----	150.6	149.8	1007	1743	1573	22.8	16.9	25.2
2	146.0	150.9	149.5	972	1802	1766	20.8	13.2	28.5
3	145.4	149.8	149.7	852	1833	1487	19.4	13.9	27.4
Average	145.7	150.4	149.7	944	1793	1609	21.0	14.7	27.0

(continued)

Table 24 (continued)

Summary of Bulk Density and Marshall Test Results
(1954, 1955, 1957)

Specimen Number	Bulk Density (lb./cu.ft.) 1954	1955	1957	Marshall Stability (lbs.) 1954	1955	1957	Marshall Flow (1/100 in.) 1954	1955	1957
41-1-WA	147.3	150.0	150.0	1529	2079	2350	24.2	23.1	20.3
2	146.6	151.4	150.5	----	2479	2531	24.6	23.2	16.4
3	147.9	151.4	150.8	1677	1966	2639	22.8	28.1	21.4
Average	147.2	150.9	150.4	1603	2175	2507	23.9	24.8	19.5
41-1-OA	145.4	151.1	153.3	1302	2139	2637	27.4	19.4	16.6
2	146.0	150.5	152.7	1439	2184	2907	24.0	21.3	16.4
3	146.0	151.3	152.6	1134	2173	2810	27.4	26.5	14.0
Average	145.8	151.0	152.9	1292	2165	2785	26.3	22.4	15.7
41-1-WB	142.3	148.5	146.7	816	1588	1709	28.9	18.4	19.5
2	142.9	148.5	149.4	865	1657	2183	29.3	23.5	25.9
3	142.3	147.8	150.0	---	1493	2573	----	24.5	27.2
Average	142.5	148.3	148.7	841	1579	2155	29.1	22.1	24.2
41-1-OB	142.3	147.7	147.2	740	1639	1617	22.7	18.7	31.5
2	142.3	147.6	147.7	787	1826	1743	20.6	23.9	24.6
3	143.5	148.8	146.5	955	1579	1633	24.9	16.8	21.0
Average	142.7	148.0	147.1	827	1681	1664	22.7	19.8	25.7
41-1-WC	142.3	146.6	146.0	938	1865	1569	25.1	24.2	22.2
2	-----	147.9	145.3	948	1507	1882	20.4	20.2	22.6
3	142.3	147.5	145.0	1029	1631	1605	25.1	22.5	21.9
Average	142.3	147.3	145.4	972	1668	1685	23.5	22.3	22.2
41-1-OC	141.0	146.1	146.5	1040	----	2331	48.8	----	26.4
2	142.9	145.5	147.5	1110	1904	2027	31.9	30.2	20.4
3	142.3	145.6	148.5	1023	1501	2064	39.5	25.6	32.2
4	142.9	----	----	----	----	----	----	----	----
Average	142.3	145.7	147.5	1058	1703	2141	40.1	27.9	26.3

(continued)

Table 24 (continued)
Summary of Bulk Density and Marshall Test Results
(1954, 1955, 1957)

Specimen Number	Bulk Density (lb./cu.ft.) 1954	1955	1957	Marshall Stability (lbs.) 1954	1955	1957	Marshall Flow (1/100 in.) 1954	1955	1957
12-1-WA	144.8	151.8	153.1	860	1883	1929	17.3	14.9	13.2
2	146.0	151.1	153.0	749	1194	1679	20.5	10.8	21.3
3	144.8	151.7	153.4	501	1690	1957	12.5	----	22.4
Average	145.2	151.5	153.2	703	1589	1855	16.8	12.9	19.0
12-1-OA	147.9	151.8	152.8	1018	1460	1568	16.7	19.9	15.1
2	148.5	151.7	152.8	1018	1577	2082	----	15.8	14.9
3	148.5	150.9	152.6	----	1452	1845	14.5	12.0	14.6
Average	148.3	151.4	152.7	1018	1496	1832	15.6	15.9	14.9
12-1-WB	147.9	150.1	152.6	802	1428	1512	16.8	21.6	13.8
2	147.9	150.1	152.1	804	1420	1608	----	14.8	12.7
3	147.3	150.7	152.4	566	1428	1420	11.2	14.6	11.6
4	146.6	----	----	658	----	----	9.8	----	----
5	147.9	----	----	692	----	----	15.9	----	----
Average	147.5	150.3	152.4	704	1425	1513	13.4	17.0	12.7
12-1-OB	142.9	146.9	147.0	662	843	----	13.8	11.6	----
2	144.1	147.3	146.3	743	1325	986	13.6	12.3	13.2
3	144.8	147.1	147.3	508	1147	1002	8.8	12.5	16.5
Average	143.9	147.1	146.9	638	1105	994	12.1	12.1	14.9
12-1-WC	148.5	150.7	151.2	821	1569	1222	9.1	10.5	18.3
2	147.9	147.5	150.9	873	1559	1248	13.9	12.2	21.9
3	147.9	150.7	151.3	756	----	1620	----	13.9	17.0
Average	148.1	149.4	151.1	817	1564	1363	11.5	12.2	19.1
12-1-OC	144.1	147.3	147.9	529	1196	1009	13.1	9.9	13.0
2	140.4	148.1	148.5	466	1153	832	----	11.0	18.2
3	142.3	148.6	148.8	744	1440	----	16.4	12.5	----
Average	142.2	148.0	148.4	580	1263	921	15.8	11.1	15.4

Table 25

Asphalt Content by Extraction - Rice Specific Gravity Analysis (a)

Specimen	Percent By Wt. of Agg.	Percent By Wt. of Mix	Maximum Specific Gravity	Bulk Specific Gravity (Compacted)	Percent Voids Total Mix
20-0A Group					
Surface	6.4	6.0	----	----	----
Binder	5.1	4.8	----	----	----
20-1WA	----	----	2.48	2.44	1.6
20-0B Group					
Surface	6.9	6.4	----	----	----
Binder	4.8	4.6	----	----	----
20-1WB	----	----	2.46	2.40	2.4
20-0C Group					
Surface	9.9	9.0	----	----	----
Binder	5.9	5.6	----	----	----
20-1WC	----	----	2.46	2.42	1.6
41-0A Group					
Surface	7.0	6.6	----	----	----
Binder	6.2	5.9	----	----	----
41-1WA	----	----	2.51	2.41	4.0
41-0B Group					
Surface	8.1	7.5	----	----	----
Binder	6.3	5.9	----	----	----
41-1WB	----	----	2.46	2.35	4.5
41-0C Group					
Surface	6.8	6.4	----	----	----
Binder	5.8	5.5	----	----	----
41-1WC	----	----	2.47	2.35	4.9

(a) Samples were taken in 1957 after three years of service.

(continued)

Table 25 (continued)

Asphalt Content by Extraction - Rice Specific Gravity Analysis (a)

Specimen	Percent By Wt. of Agg.	Percent By Wt. of Mix	Maximum Specific Gravity	Bulk Specific Gravity (Compacted)	Percent Voids Total Mix
12-0A Group					
Surface	6.3	5.9	----	----	----
Binder	5.8	5.4	----	----	----
12-1WA	----	----	2.47	2.45	0.8
12-0B Group					
Surface	6.2	5.8	----	----	----
Binder	5.6	5.3	----	----	----
12-1WB	----	----	2.46	2.44	0.8
12-0C Group					
Surface	6.5	6.1	----	-----	----
Binder	5.2	4.9	----	-----	----
12-1WC	----	----	2.46	2.42	1.6

(a) Samples were taken in 1957 after three years of service.

Table 26

Sieve Analyses (Percent Passing) (a)

Sieve Size	20-0A Surface	20-0A Binder	20-0B Surface	20-0B Binder	20-0C Surface	20-0C Binder	41-0A Surface	41-0A Binder	41-0B Surface	41-0B Binder
1	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
3/4	100.0	100.0	100.0	96.9	100.0	97.3	100.0	96.4	100.0	97.3
1/2	100.0	81.8	100.0	75.2	100.0	79.9	87.8	75.7	100.0	70.9
3/8	90.6	59.4	92.6	52.3	90.2	58.6	76.1	51.2	92.0	48.4
4	50.6	42.3	51.8	35.1	50.1	41.2	49.4	37.9	59.0	34.3
8	37.2	36.6	36.9	30.1	36.8	34.1	38.1	32.5	41.7	29.3
16	30.7	28.5	30.5	23.5	30.7	26.4	28.2	24.2	30.7	22.4
30	21.9	20.8	21.4	17.1	22.1	19.2	19.7	17.3	21.5	16.3
50	8.5	14.9	8.5	11.9	7.7	13.0	14.2	12.7	15.5	12.3
100	3.7	10.6	3.7	8.1	2.9	9.0	10.2	8.9	11.4	9.2
200	2.9	6.5	2.9	5.0	2.3	6.2	6.6	5.5	8.1	6.6
Pan	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Sieve Size	41-0C Surface	41-0C Binder	12-0A Surface	12-0A Binder	12-0B Surface	12-0B Binder	12-0C Surface	12-0C Binder
1	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
3/4	100.0	98.5	100.0	100.0	100.0	100.0	100.0	100.0
1/2	97.6	80.5	97.2	88.2	93.4	88.7	100.0	92.0
3/8	92.3	59.7	89.2	67.4	69.6	63.9	92.8	67.1
4	56.6	40.8	54.0	43.9	42.0	37.1	54.0	40.0
8	39.4	32.4	38.4	32.9	30.6	26.9	37.5	28.9
16	30.0	25.2	32.0	27.6	26.2	22.8	31.5	24.3
30	21.4	18.5	24.0	19.9	19.6	16.9	23.6	17.8
50	13.7	13.6	10.2	7.2	7.7	6.4	9.5	7.0
100	9.1	9.9	4.7	3.0	3.3	2.5	4.0	2.9
200	6.7	7.1	3.8	2.5	2.7	1.8	3.5	2.4
Pan	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

(a) Samples were taken in 1957 after three years of service.

APPENDIX E

DATA - 1959

APPENDIX E

DATA - 1959

Complete test results for the samples taken in 1959 from each of the three pavements are tabulated in this section. Test results for laboratory mixtures using the standard compactor spring are also presented. These results are summarized and presented in graphical form in the discussion of results section of this report.

The tabulation of complete test results is presented here for the data record and further analysis and evaluation of these data, if desired.

Table 27

1959 Data
Composite-Core Test Results

Specimen Ident.	Total Height, in.	Marshall Stability, lbs.	Marshall Flow, 1/100 in.	Hveem Stability	Bulk Density, pcf	Rice Maximum Density, pcf	Percent Mix Voids
20-WA	2.34	2467	15.3	22.1	153.8		1.7
20-WA	2.34	2455	---	25.0	153.6	156.5	1.9
20-WA	2.32	2199	15.6	25.0	152.8		2.4
20-WB	2.28	1983	15.8	19.3	151.1		2.3
20-WB	2.27	1862	15.2	22.6	150.8	154.7	2.5
20-WB	2.22	---	---	23.4	150.6		2.7
20-WC	2.58	1572	19.2	24.8	150.2		5.1
20-WC	2.74	1525	13.1	25.1	150.0	158.2	5.2
20-WC	2.74	1470	16.6	26.0	149.9		5.3
20-OA	2.13	2198	18.9	---	152.5		2.1
20-OA	2.05	2472	16.5	---	152.4	155.8	2.2
20-OA	2.05	2094	18.0	---	152.9		1.9
20-OB	2.14	2343	16.4	---	150.6		2.4
20-OB	2.16	1703	22.2	---	149.9	154.3	2.9
20-OB	2.15	1776	19.0	---	150.6		2.4
20-OC	2.49	1643	9.9	23.9	150.8		2.8
20-OC	2.52	1719	9.5	28.6	150.9	155.2	2.8
20-OC	2.54	1556	16.1	21.9	148.3		4.5
41-WA	2.74	2029	16.2	22.5	152.3		2.6
41-WA	2.87	---	---	17.6	152.4	156.4	2.6
41-WA	2.82	2559	14.2	15.8	152.6		2.4
41-WB	3.32	1552	24.9	14.0	148.4		1.8
41-WB	3.31	1625	18.6	12.1	148.7	151.1	1.6
41-WB	3.22	1440	28.8	12.4	147.5		1.7
41-WC	3.13	1649	20.9	13.9	147.5		3.1
41-WC	3.02	1892	25.4	14.0	147.3	152.2	3.2
41-WC	2.89	1635	26.7	12.0	145.6		4.3
41-OA	3.09	1985	21.9	20.9	150.7		3.1
41-OA	3.03	2018	23.6	23.2	151.8	155.5	2.4
41-OA	3.29	1755	19.0	23.1	151.1		2.8

(Continued)

Table 27 (Continued)

1959 Data
Composite-Core Test Results

Specimen Ident.	Total Height, in.	Marshall Stability, lbs.	Marshall Flow, 1/100 in.	Hveem Stability	Bulk Density pcf	Rice Maximum Density, pcf	Percent Mix Voids
41-OB	2.90	2123	23.5	13.4	150.0		2.5
41-OB	2.79	2017	22.9	16.2	149.6	153.9	2.8
41-OB	3.01	1700	24.3	14.0	148.8		3.3
41-OC	3.65	---	---	14.5	146.5		4.9
41-OC	3.30	1989	35.9	14.5	147.7	154.0	4.1
41-OC	3.51	1820	26.5	16.2	146.0		5.2
12-WA	2.68	1737	16.7	15.4	153.5		1.4
12-WA	2.68	1458	16.6	12.0	153.3	155.7	1.5
12-WA	2.70	1377	14.6	12.0	153.7		1.3
12-WB	1.49	1099	19.2	---	152.2		0.5
12-WB	1.52	1025	18.7	---	152.5	152.9	0.3
12-WB	1.57	1053	15.2	---	152.6		0.2
12-WC	2.21	1626	15.9	---	151.4		2.1
12-WC	2.24	1740	16.0	---	150.9	154.6	2.4
12-WC	2.14	1688	---	---	152.0		1.7
12-OA	3.09	2279	11.0	27.1	152.2		2.5
12-OA	3.09	1937	10.3	27.5	152.8	156.1	2.1
12-OA	3.11	1810	15.3	26.1	152.7		2.2
12-OB	2.04	596	11.4	---	147.5		4.7
12-OB	1.99	----	11.0	---	146.8	154.8	5.2
12-OB	2.10	527	14.4	---	146.3		5.5
12-OC	2.52	1057	9.7	27.8	149.6		3.2
12-OC	2.57	861	13.1	28.0	149.8	154.6	3.1
12-OC	2.55	842	11.5	28.9	149.4		3.4

Table 28

1959 Data
Surface-Sample Test Results

Specimen Identification	Surface Thickness in.	Hveem Stability	Bulk Density, pcf	Rice Maximum Density, pcf	Percent Mix Voids	Percent Asphalt Content in Mix by Extraction
20-WA	0.91		153.3		---	
20-WA	0.86	15.9	153.4	---	---	7.2, 5.9
20-WA	0.85		153.4		---	
20-WB	0.84		151.9		---	
20-WB	0.84	19.1	151.8	---	---	6.6
20-WB	0.84		152.2		---	
20-WC	0.89		---		---	
20-WC	0.87	25.3	151.2	---	---	5.9, 7.0
20-WC	0.91		151.2		---	
20-OA	0.95		153.0		0.8	
20-OA	1.10	34.8	153.1	154.3	0.9	---
20-OA	0.97		153.6		0.5	
20-OB	1.05		151.5		---	
20-OB	0.97	23.2	151.3	---	---	---
20-OB	1.02		151.7		---	
20-OC	0.71		150.9		1.4	
20-OC	0.83	28.9	150.5	153.0	1.6	---
20-OC	0.78		151.1		1.2	
41-WA	0.61		152.9		---	
41-WA	0.68	5.4 (a)	153.6	---	---	7.8, 6.7
41-WA	0.58		154.0		---	
41-WB	0.68		152.6		---	
41-WB	0.79	---	---	---	---	7.4
41-WB	0.70		151.7		---	
41-WC	0.74		151.9		---	
41-WC	0.82	10.0	---	---	---	6.8, 7.1
41-WC	0.76		151.6		---	

(a) Results extrapolated when lateral pressure exceeded maximum dial reading of 200 psi.

(Continued)

Table 28 (Continued)

1959 Data
Surface-Sample Test Results

Specimen Identification	Surface Thickness, in.	Hveem Stability	Bulk Density, pcf	Rice Maximum Density, pcf.	Percent Mix Voids	Percent Asphalt Content in Mix by Extraction
41-OA	0.39		154.5		1.1	
41-OA	0.49	---	153.8	156.2	1.5	---
41-OA	0.48		153.4		1.8	
41-OB	0.69		151.9		---	
41-OB	0.72	---	152.6	---	---	---
41-OB	---		---		---	
41-OC	0.81		151.1		0.9	
41-OC	0.85	---	151.1	152.5	0.9	---
41-OC	---		---		---	
12-WA	0.73		151.3		---	
12-WA	0.77	---	154.3	---	---	5.2, 5.7
12-WA	0.66		154.5		---	
12-WB	---		---		---	
12-WB	0.25	---	152.4	---	---	5.6
12-WB	0.52		151.3			
12-WC	0.83		152.2		---	
12-WC	0.84	---	152.0	---	---	6.4, 6.0
12-WC	0.69		152.1		---	
12-OA	0.81		152.3		2.0	
12-OA	0.87	34.0	152.6	155.4	1.8	---
12-OA	0.90		152.0		2.2	
12-OB	---		---		---	
12-OB	0.34	---	151.6	---	---	---
12-OB	0.31		151.6		---	
12-OC	0.81		151.3		1.4	
12-OC	0.78	---	151.4	153.4	1.3	---
12-OC	0.73		151.5		1.2	

Table 29

1959 Data
Surface-Sample Aggregate Gradations

Sieve Size Passing	Sieve Size Retained	20-WA		20-WB	20-WC	
1/2 in.	3/8 in.	7.1	7.0	9.1	6.0	8.7
3/8 in.	No. 4	39.8	39.0	37.0	37.5	32.9
No. 4	No. 6	9.6	10.1	10.1	10.6	9.4
No. 6	No. 8	5.1	5.7	6.0	4.7	6.2
No. 8	No. 16	7.2	7.3	7.2	8.6	11.9
No. 16	No. 50	20.8	21.0	20.5	22.7	15.4
No. 50	No. 100	5.6	5.6	5.7	4.9	4.7
No. 100	No. 200	1.1	0.8	0.8	0.8	3.2
No. 200	---	3.7	3.5	3.6	4.2	7.6
Total retained on No. 6 by wt. of mix		53.2	52.1	52.5	50.9	47.4
Sample weight, gms.		1139	1084	1126	1096	1010

(Percent Between; Sample Identification)

Sieve Size Passing	Sieve Size Retained	41-WA		41-WB	41-WC	
1/2 in.	3/8 in.	8.1	7.0	6.3	6.3	7.4
3/8 in.	No. 4	36.7	34.7	34.0	33.2	35.9
No. 4	No. 6	8.8	10.2	10.3	10.4	10.6
No. 6	No. 8	5.4	6.0	7.0	6.4	6.1
No. 8	No. 16	11.0	11.8	12.2	12.3	7.7
No. 16	No. 50	15.3	15.3	15.6	15.5	22.7
No. 50	No. 100	5.8	4.4	4.5	4.5	5.5
No. 100	No. 200	2.4	3.0	3.2	3.2	0.7
No. 200	---	6.7	7.6	6.9	8.2	3.4
Total retained on No. 6 by wt. of mix		49.4	48.4	46.9	46.4	50.2
Sample weight, gms.		981	757	892	951	954

(Percent Between; Sample Identification)

(Continued)

Table 29 (Continued)

1959 Data
Surface-Sample Aggregate Gradations

Sieve Size Passing	Sieve Size Retained	Percent Between, Sample Identification: 12-WA	12-WA	12-WB	12-WC	12-WC
1/2 in.	3/8 in.	7.4	6.0	6.2	5.0	8.2
3/8 in.	No. 4	39.5	38.6	40.4	38.9	40.5
No. 4	No. 6	9.9	10.2	10.2	10.1	9.4
No. 6	No. 8	5.3	5.5	5.4	5.5	4.7
No. 8	No. 16	6.8	7.6	17.0	7.3	6.3
No. 16	No. 50	19.9	21.8	10.5	22.8	21.8
No. 50	No. 100	5.1	5.5	5.7	5.7	5.2
No. 100	No. 200	0.9	0.9	1.1	0.9	0.9
No. 200	---	5.2	3.9	3.5	3.8	3.0
Total retained on No. 6 by wt. of mix		53.9	51.7	53.6	50.5	54.6
Sample weight, gms.		811	911	339	668	963

Table 30

1959 Data
Binder-Sample Test Results

Specimen Identification	Binder Thickness, in.	Hveem Stability	Bulk Density, pcf	Percent Asphalt Content in Mix by Extraction
20-WA	1.48		154.2	
20-WA	1.49	29.3	153.8	5.6
20-WA	1.52		153.6	
20-WB	1.42		152.1	
20-WB	1.40	24.7	152.6	6.0
20-WB	1.38		152.8	
20-WC	1.64		152.8	
20-WC	1.88	28.0	153.8	5.6
20-WC	1.81		152.8	
20-OA	1.15		---	
20-OA	0.90	29.0	150.2	---
20-OA	1.06		---	
20-OB	1.07		146.5	
20-OB	1.18	22.0	146.3	---
20-OB	1.13		148.9	
20-OC	1.81		---	
20-OC	1.70	23.4	---	---
20-OC	1.77		---	
41-WA	2.12		152.2	
41-WA	1.99	25.9	153.7	5.8
41-WA	2.24		153.5	
41-WB	2.57		149.4	
41-WB	2.26	15.2	149.1	5.8
41-WB	2.66		149.2	
41-WC	2.43		148.8	
41-WC	2.24	22.4	148.8	5.6
41-WC	2.26		149.4	

(Continued)

Table 30 (Continued)

1959 Data
Binder-Sample Test Results

Specimen Identification	Binder Thickness, in.	Hveem Stability	Bulk Density, pcf	Percent Asphalt Content in Mix by Extraction
41-OA	2.88		150.4	
41-OA	2.67	36.6	150.6	---
41-OA	3.02		150.9	
41-OB	1.99		147.8	
41-OB	2.23	18.5	147.5	---
41-OB	---		147.6	
41-OC	2.96		147.5	
41-OC	2.45	21.6	146.8	---
41-OC	---		146.5	
12-WA	1.72		153.1	
12-WA	1.84	24.0	153.1	6.4
12-WA	1.97		152.9	
12-WB	---		149.8	
12-WB	1.25	40.6	---	5.7
12-WB	1.03		151.4	
12-WC	1.34		152.2	
12-WC	1.39	33.6	151.9	6.2
12-WC	1.40		151.3	
12-OA	2.27		152.1	
12-OA	2.20	48.7	151.7	---
12-OA	2.21		151.5	
12-OB	---		150.9	
12-OB	1.66	27.8	150.5	---
12-OB	1.76		151.3	
12-OC	1.77		148.3	
12-OC	1.80	35.4	148.4	---
12-OC	1.84		148.6	

Table 31

1959 Data
Binder-Sample Aggregate Gradations

Sieve Size Passing	Sieve Size Retained	Percent Between Sample Identification 20-WA	20-WB	20-WC
1 in.	1/2 in.	19.2	16.5	19.8
1/2 in.	No. 4	42.6	44.3	42.0
No. 4	No. 6	1.9	1.8	2.0
No. 6	No. 8	2.8	2.7	2.8
No. 8	No. 16	7.9	8.9	8.6
No. 16	No. 50	11.7	12.7	12.4
No. 50	No. 100	3.8	3.6	3.5
No. 100	No. 200	3.3	3.1	2.8
No. 200	---	6.8	6.4	6.1
Total retained on No. 6 by wt. of mix		60.2	58.8	60.2
Sample weight, gms.		1116	1166	1104

Sieve Size Passing	Sieve Size Retained	Percent Between Sample Identification 41-WA	41-WB	41-WC
1 in.	1/2 in.	31.0	26.0	29.9
1/2 in.	No. 4	31.4	37.6	37.5
No. 4	No. 6	3.7	3.6	2.9
No. 6	No. 8	2.7	2.5	2.3
No. 8	No. 16	8.1	7.3	7.3
No. 16	No. 50	11.7	10.9	9.4
No. 50	No. 100	4.0	3.7	3.1
No. 100	No. 200	2.1	2.2	2.2
No. 200	---	5.3	6.2	5.4
Total retained on No. 6 by wt. of mix		62.3	63.3	66.4
Sample weight, gms.		1083	964	1048

(Continued)

Table 31 (Continued)

1959 Data
Binder-Sample Aggregate Gradations

Sieve Size		Percent Between		
		Sample Identification		
Passing	Retained	12-WA	12-WB	12-WC
3/4 in.	1/2 in.	9.5	8.7	7.6
1/2 in.	No. 4	47.3	50.5	45.8
No. 4	No. 6	6.4	6.2	7.4
No. 6	No. 8	4.2	3.3	4.9
No. 8	No. 16	5.7	5.5	6.3
No. 16	No. 50	19.6	19.1	20.7
No. 50	No. 100	4.3	4.2	4.7
No. 100	No. 200	0.7	0.7	0.7
No. 200	---	2.3	1.8	1.9
Total retained on No. 6 by wt. of mix		59.2	61.7	57.1
Sample weight, gms.		1092	1000	1108

Table 32

Recovered-Asphalt Test Results
Surface Samples - 1959

Specimen Identification	Test					
	Penetration, 100 gm, 5 sec, 77°F	Softening Point, R & B, °F	Ductility, 77°F, cm (a)			
			Trial 1	Trial 2	Trial 3	Average
20-WA	43	132	105+	110+	95+	103+
20-WC	43	130	90+	90+	90+	90+
41-WA	47	137	95	120	---	108
41-WC	34	133	141	150+	---	146+
41-WC	45	136	95	105	110	103
12-WA	52	128	90+	100+	---	95+
12-WA	45	127	110+	120+	---	115+
12-WC	46	129	120	150+	---	135+
12-WC	45	131	94	136	---	115

(a) Ductility values less than 150 cm and followed by a plus sign indicate the asphalt sample made contact with the water bath surface or tank bottom at the recorded value.

Table 33

Gradation "A" Aggregate and Mixture Properties

Sample	Specific Gravity of Coarse Aggregate		Absorption, Percent
	Bulk	Apparent	
1	2.58	2.72	1.92
2	2.59	2.72	1.84
3	2.59	2.72	1.82
Average	2.59	2.72	1.86

Sample	Specific Gravity of Fine Aggregate		Absorption, Percent
	Bulk	Apparent	
1	2.70	2.78	1.09
2	2.69	2.78	1.13
3	2.69	2.75	0.71
Average	2.69	2.77	0.98

Surface area, sq ft / lb	=	28.5
CKE, percent	=	3.4 (Av. of two)
OE, percent	=	3.5 (Av. of two)
Optimum A/C, percent aggregate weight	=	5.8
Design A/C (by mix), percent	=	7.0
Design A/C (by agg.), percent	=	7.6
Rice specific gravity	=	2.45 (Av. of three)
Theo. maximum specific gravity (Bulk)	=	2.38
Theo. maximum specific gravity (App.)	=	2.46
Theo. maximum specific gravity (Eff.)	=	2.42

Table 34

Gradation "B" Aggregate and Mixture Properties

Sample	Specific Gravity of Coarse Aggregate		Absorption, Percent
	Bulk	Apparent	
1	2.62	2.71	2.05
2	2.63	2.72	2.03
3	2.63	2.71	1.95
Average	2.63	2.71	2.01

Sample	Specific Gravity of Fine Aggregate		Absorption, Percent
	Bulk	Apparent	
1	---	---	---
2	2.56	2.60	0.97
3	2.57	2.61	0.97
Average	2.57	2.61	0.97

Specific gravity of mineral filler	= 2.72 (Av. of three)
Surface area, sq ft/lb	= 17.9
CKE, percent	= 1.9 (Av. of two)
OE, percent	= 3.5 (Av. of two)
Optimum A/C, percent aggregate weight	= 5.5
Rice Specific gravity	= 2.42
Design A/C (by mix), percent	= 6.0
Design A/C (by agg.), percent	= 6.3
Theo. maximum specific gravity (Bulk)	= 2.38
Theo. maximum specific gravity (App.)	= 2.43
Theo. maximum specific gravity (Eff.)	= 2.41

Table 35

Gradation "C" Aggregate and Mixture Properties

Sample	Specific Gravity of Coarse Aggregate		Absorption, Percent
	Bulk	Apparent	
1	2.56	2.70	2.09
2	2.56	2.70	2.02
3	2.56	2.70	2.05
Average	2.56	2.70	2.05

Sample	Specific Gravity of Fine Aggregate		Absorption, Percent
	Bulk	Apparent	
1	2.66	2.79	1.77
2	2.67	2.76	1.22
3	2.68	2.78	1.40
Average	2.67	2.78	1.46

Surface area, sq ft/lb	= 24.7	
CKE, percent	= 4.0	(Av. of two)
OE, percent	= 3.9	(Av. of two)
Optimum A/C, percent agg. wt.	= 5.8	
Design A/C (by mix), percent	= 5.7	
Design A/C (by agg.), percent	= 6.0	
Rice specific gravity	= 2.44	(Av. of two)
Theo. maximum specific gravity (Bulk)	= 2.39	
Theo. maximum specific gravity (App.)	= 2.49	
Theo. maximum specific gravity (Eff.)	= 2.44	

Table 36

Gradation "D" Aggregate and Mixture Properties

Sample	Specific Gravity of Coarse Aggregate		Absorption, Percent
	Bulk	Apparent	
1	2.56	2.70	1.93
2	2.56	2.71	2.16
3	2.56	2.70	2.05
Average	2.56	2.70	2.05

Sample	Specific Gravity of Fine Aggregate		Absorption, Percent
	Bulk	Apparent	
1	2.47	2.55	1.24
2	---	2.63	1.01
3	2.49	2.60	---
Average	2.48	2.59	1.12

Surface area, sq ft/lb	= 10.4
CKE, percent	= 1.9 (Av. of two)
OE, percent	= 3.2 (Av. of two)
Optimum A/C , percent agg. wt.	= 5.8
Design A/C (by mix), percent	= 6.1
Design A/C (by agg.), percent	= 6.5
Rice specific gravity	= 2.41 (Av. of three)
Theo. maximum specific gravity (Bulk)	= 2.33
Theo. maximum specific gravity (App.)	= 2.43
Theo. maximum specific gravity (Eff.)	= 2.38

Table 37

Routine Test Results for Laboratory Compacted Specimens
(Standard Spring)

Gradation	Compaction Pressure, psi	Hveem Stability	Bulk Density, pcf	Percent Mix Voids (1)
A	70	31.2	146.1	4.3
A	125	28.9	150.0	1.8
A	225	22.7	152.4	0.2
A	275	27.8	151.5	0.8
A	330	26.8	152.4	0.2
A	385	14.7	152.8	---
A	460	1.7(2)	153.2	---
A	50-blow Marshall	39.6	147.1	3.7
A	50-blow Marshall	41.0	148.0	3.1
A	50-blow Marshall	42.4	148.0	3.1
B	275	35.6	148.4	1.7
B	460	37.6	149.0	1.3
B	595	32.1	150.4	0.3
B	645	31.8	150.4	0.3
B	50-blow Marshall	----	----	---
B	50-blow Marshall	27.3	142.1	5.8
B	50-blow Marshall	26.2	140.3	7.0
C	125	21.8	146.3	3.7
C	275	32.9	148.3	2.4
C	385	38.9	149.9	1.3
C	460	32.8	151.2	0.5
C	525	46.3	151.1	0.5
C	595	42.9	151.8	0.1
C	50-blow Marshall	22.6	147.9	2.6
C	50-blow Marshall	28.6	147.6	2.8
C	50-blow Marshall	23.4	149.2	1.8
D	125	31.6	145.3	3.4
D	175	34.1	145.6	3.2
D	275	41.0	146.5	2.6
D	385	41.9	147.8	1.7
D	460	38.3	149.1	0.9
D	525	44.4	148.8	1.1
D	595	46.2	149.2	0.8
D	645	42.1	149.4	0.7

(Continued)

Table 37 (continued)

Routine Test Results for Laboratory Compacted Specimens
(Standard Spring)

Gradation	Compaction Pressure , psi	Hveem Stability	Bulk Density, pcf	Percent Mix Voids (1)
D	50-blow Marshall	23.2	142.8	5.1
D	50-blow Marshall	18.5	140.8	6.4
D	50-blow Marshall	30.6	142.5	5.3

(1) Determined by use of Rice maximum density.

(2) Results extrapolated when the lateral pressure exceeded the maximum dial reading of 200 psi.

Table 38

Specimen Uniformity Test Results (Standard Spring)

Gradation	Compaction Pressure, psi	Layer	Paraffin-Coated * Bulk Density, pcf	Extracted Asphalt Content, Percent by Mix Weight	Percent Aggregate Retained on No. 6 by Mix Weight
A	275	top	155.3	---	---
A	275	middle	152.4	---	---
A	275	bottom	148.9	---	---
A	330	top	154.9	6.4	47.7
A	330	middle	152.4	6.9	48.8
A	330	bottom	147.5	8.0	44.0
A	385	top	152.8	---	---
A	385	middle	152.0	---	---
A	385	bottom	147.9	---	---
A	460	top	152.8	---	---
A	460	middle	154.5	---	---
A	460	bottom	151.4	---	---
B	275	top	150.6	---	---
B	275	middle	148.9	---	---
B	275	bottom	146.1	---	---
B	460	top	150.8	6.3	40.5
B	460	middle	148.4	6.4	45.5
B	460	bottom	146.3	7.0	46.8
B	595	top	150.3	---	---
B	595	middle	150.8	---	---
B	595	bottom	147.5	---	---
B	645	top	150.6	---	---
B	645	middle	149.9	---	---
B	645	bottom	147.9	---	---
C	385	top	151.4	---	---
C	385	middle	151.6	---	---
C	385	bottom	148.2	---	---
C	460	top	153.7	---	---
C	460	middle	151.2	---	---
C	460	bottom	148.6	---	---

(Continued)

Table 38 (continued)

Specimen Uniformity Test Results (Standard Spring)

Gradation	Compacted Pressure, psi	Layer	Paraffin-Coated Bulk Density pcf	Extracted Asphalt Content, Percent by Mix Weight	Percent Aggregate Retained on No. 6 by Mix Weight
C	525	top	153.3	6.1	60.2
C	525	middle	152.4	6.0	61.1
C	525	bottom	149.0	6.5	68.6
C	595	top	154.2	---	---
C	595	middle	152.3	---	---
C	595	bottom	150.7	---	---
D	460	top	150.6	---	---
D	460	middle	149.5	---	---
D	460	bottom	146.7	---	---
D	525	top	149.3	6.4	53.0
D	525	middle	148.7	6.2	61.0
D	525	bottom	143.8	6.6	62.2
D	595	top	150.1	---	---
D	595	middle	148.1	---	---
D	595	bottom	144.5	---	---
D	645	top	148.2	---	---
D	645	middle	148.1	---	---
D	645	bottom	144.3	---	---

APPENDIX F

DATA - OLD COMPACTOR SPRING

APPENDIX F

DATA - OLD COMPACTOR SPRING

Numerous specimens were molded and tested for each of the four laboratory mixtures using the old (low-capacity) spring installed in the kneading compactor. Complete test results are presented here for the data record. Further analysis of these data may be desirable, especially of the aggregate degradation data. For this reason, the data tabulated are presented in full rather than as average values.

A summary of the data and graphical presentation of the results are shown in Appendix G of this report.

Table 39

Remolded-Sample Test Results (1),(Old Spring)

Layer	Specimen Identification	Bulk Density, pcf	Hveem Stability
	Specimens Remolded From Tested Pavement Cores		
Surface	20-OA	153.4	16.2
	20-OB	152.4	4.4 (2)
	20-OC	153.0	5.7 (2)
	12-OA	154.6	17.7
Binder	20-OA	155.8	25.6
	20-OB	154.4	35.8
	20-OC	156.2	24.0
	41-OA	152.0	40.1
	41-OB	153.3	25.0
	41-OC	155.1	29.2
	12-OA	155.4	20.2
	12-OB	154.8	14.5
	12-OC	153.3	39.6
	Core and Remolded Samples From the Same Pavement Section and Position		
Surface	20-WB	152.8	5.0 (2)
	20-OB	152.4	4.4 (2)
Binder	20-WA	154.3	39.4
	20-WB	154.6	32.2
	41-WA	153.3	58.0
	41-WB	153.6	37.9
	41-WC	151.9	40.6
	41-OA	152.8	52.7
	41-OC	151.8	37.6
	12-WA	152.2	49.0
	12-WB	152.8	35.7
	12-WC	155.5	35.6
	12-WC	154.8	19.5

(Continued)

Table 39 (Continued)

Remolded-Sample Test Results (1), (Old Spring)

Layer	Specimen Identification	Bulk Density, pcf	Hveem Stability
	12-0A	152.5	34.0
	12-0B	153.6	14.8
	12-0B	151.9	39.1
	12-0B	153.9	36.6
	12-0C	151.6	39.8

(1) Remolding compaction pressure = 595 psi.

(2) Results extrapolated when lateral pressure exceeded maximum dial reading of 200 psi.

Table 40

Gradation "A" Marshall Test Results
(Old Spring)

Compaction Pressure, psi	Test Property				
	Marshall Stability, lbs	Marshall Flow, 1/100-in.	Bulk Density, pcf	Percent Mix Voids (1)	Percent Agg. Voids filled (2)
70	2025	19.2	147.9	3.1	69.1
125	1978	20.3	147.8	3.2	68.6
225	2491	19.6	148.6	2.7	72.2
275	2483	20.4	149.9	2.5	73.8
330	2970	16.4	150.3	1.6	81.7
385	2944	19.6	151.3	0.9	88.4
460	3198	19.4	152.0	0.5	93.9
Marshall Compacted	494	---	148.4	2.8	71.3
	467	2.4	147.8	3.2	68.6
	542	2.3	148.0	3.1	69.5

(1) Rice maximum density value of 152.7 used to compute voids.

(2) Design asphalt content of 7.0 percent used to compute percent voids filled with asphalt.

Table 41

Gradation "A" Hveem Test Results
(Old Spring)

Compaction Pressure, psi	Test Property		
	Hveem Stability	Bulk Density, pcf	Percent Mix Voids (1)
70	28.4	146.4	4.1
70	33.3	148.3	3.5
70	27.2	147.8	3.2
125	35.2	148.4	3.5
125	35.1	148.6	2.7
125	34.2	147.7	3.3
225	35.5	150.1	1.7
225	36.4	150.4	1.5
225	37.9	150.6	1.4
275	40.9	150.1	1.7
275	35.6	150.0	1.8
275	34.0	150.8	1.2
330	33.7	151.5	0.8
330	36.7	151.2	1.0
330	---	---	---
385	---	---	---
385	20.2	152.2	0.3
385	22.5	151.7	0.7
460	8.4	152.4	0.2
460	4.9 (2)	152.8	---
460	12.5	152.9	---
Marshall Compacted	39.6	147.1	3.7
	41.0	148.0	3.1
	42.4	148.0	3.1

(1) Rice Maximum Density value of 152.7 used to compute voids.
(2) Results extrapolated when lateral pressure exceeded maximum dial reading of 200 psi.

Table 42

Gradation "A" Aggregate Gradations (Old Spring)

Sieve Size Passing	Retained	Compaction Pressure, psi 70			125		
		Percent			Between		
1/2 in.	3/8 in.	8.9	9.6	9.6	10.7	8.6	9.2
3/8 in.	No. 4	29.4	27.5	30.4	28.6	30.6	31.5
No. 4	No. 6	14.3	14.2	13.0	14.7	13.7	12.2
No. 6	No. 8	5.8	6.1	5.3	5.4	5.6	5.4
No. 8	No. 16	10.9	10.9	10.9	10.5	10.5	10.3
No. 16	No. 50	16.7	17.0	16.7	17.0	17.3	17.6
No. 50	No. 100	3.9	4.3	3.9	3.6	3.8	3.8
No. 100	No. 200	3.6	3.7	3.7	3.2	3.2	3.4
No. 200	---	6.5	6.7	6.5	6.3	6.7	6.6
Total retained on No. 6 by wt. of mix		48.9	47.7	49.3	50.2	49.2	49.2

Sieve Size Passing	Retained	Compaction Pressure, psi 225			275		
		Percent			Between		
1/2 in.	3/8 in.	10.0	9.1	10.4	9.5	9.3	9.6
3/8 in.	No. 4	27.7	28.9	29.6	27.7	25.3	32.2
No. 4	No. 6	15.5	15.0	12.9	14.9	17.4	11.0
No. 6	No. 8	5.3	5.7	5.7	5.6	5.8	5.4
No. 8	No. 16	10.8	10.3	10.6	10.2	10.4	10.4
No. 16	No. 50	17.2	17.4	17.4	18.1	17.7	17.9
No. 50	No. 100	3.7	3.7	4.0	4.5	4.2	4.2
No. 100	No. 200	3.2	3.2	3.4	3.2	3.5	3.9
No. 200	--	6.6	6.7	6.0	6.3	6.4	5.4
Total retained on No. 6 by wt. of mix		49.5	49.3	49.2	48.4	48.3	49.1

(Continued)

Table 42 (continued)

Gradation "A" Aggregate Gradations (Old Spring)

Sieve Size Passing	Retained	Compaction Pressure, psi 330			385		
		Percent			Between		
1/2 in.	3/8 in.	9.0	9.1	8.5	10.0	9.9	9.1
3/8 in.	No. 4	27.6	21.3	22.2	27.7	19.5	31.3
No. 4	No. 6	16.1	22.5	21.9	14.9	23.7	13.1
No. 6	No. 8	5.6	5.5	5.7	5.9	5.3	5.8
No. 8	No. 16	10.5	10.3	10.3	10.2	10.2	10.1
No. 16	No. 50	17.3	17.1	17.3	17.3	17.3	17.2
No. 50	No. 100	3.7	3.7	3.8	4.0	3.8	3.8
No. 100	No. 200	3.4	3.3	3.2	3.5	3.4	3.0
No. 200	---	6.8	7.2	7.1	6.5	6.9	6.6
Total retained on No. 6 by wt. of mix		49.0	49.2	48.9	48.9	49.4	49.7

Sieve Size Passing	Retained	Compaction Pressure, psi 460			Marshall Compacted		
		Percent			Between		
1/2 in.	3/8 in.	10.6	10.8	9.5	8.9	10.1	10.6
3/8	No. 4	29.5	32.7	33.5	27.5	30.9	25.0
No. 4	No. 6	12.8	9.8	9.7	16.3	12.6	17.3
No. 6	No. 8	5.8	5.2	5.7	5.7	5.0	5.2
No. 8	No. 16	10.2	10.4	10.1	10.8	10.7	10.7
No. 16	No. 50	17.3	17.1	17.4	16.6	16.9	16.9
No. 50	No. 100	3.8	3.8	3.9	4.1	4.1	4.3
No. 100	No. 200	3.2	3.4	3.4	3.8	3.6	3.8
No. 200	---	6.8	6.8	6.8	6.3	6.1	6.2
Total retained on No. 6 by wt. of mix		49.2	48.5	49.0	49.0	49.8	49.2

Table 43

Gradation "B" Marshall Test Results
(Old Spring)

Compaction Pressure, psi	Test Property				
	Marshall Stability, lbs	Marshall Flow, 1/100-in.	Bulk Density, pcf	Percent Mix Voids (1)	Percent Agg. Voids filled (2)
275	2185	12.2	147.3	2.4	71.5
460	2480	14.5	149.3	1.1	84.9
595	2687	13.3	150.6	0.2	96.7
645	2879	16.5	151.6	---	---
Marshall Compacted	861	9.7	143.3	5.0	54.3
	851	7.7	143.4	5.0	54.7
	921	9.4	144.1	4.5	57.1

(1) Rice maximum density of 150.9 used to compute voids.

(2) Design asphalt content of 6.0 percent was used to compute percent voids filled with asphalt.

Table 44
Gradation "B" Hveem Test Results (Old Spring)

Compaction Pressure, psi	Test Property		
	Hveem Stability	Bulk Density, pcf	Percent Mix Voids (1)
275	34.1	147.6	2.2
275	33.1	148.8	1.4
275	36.2	148.3	1.7
460	35.3	149.4	1.0
460	32.7	149.6	0.9
460	32.6	149.8	0.7
595	17.0	151.4	---
595	17.7	151.2	---
595	---	---	---
645	16.8	150.4	0.3
645	---	---	---
645	16.0	150.8	0.1
Marshall	---	---	---
Compacted	27.3	142.1	5.8
	26.2	140.3	7.0
7.5 min. @ 595	15.0	151.1	---
7.5 min. @ 595	12.7	151.0	---
7.5 min. @ 595	18.7	150.8	0.1
10 min. @ 595	11.7	150.8	0.1
10 min. @ 595	11.0	150.8	0.1
10 min. @ 595	---	---	---
4 min. @ 525	28.5	149.5	0.9
4 min. @ 525	31.2	150.3	0.4
4 min. @ 525	---	---	---
5 min. @ 525	25.6	150.8	0.1
5 min. @ 525	24.4	149.9	0.7
5 min. @ 525	---	---	---
7 min. @ 525	21.4	150.9	0
7 min. @ 525	28.6	150.9	0
7 min. @ 525	23.3	151.3	---
8 min. @ 525	25.7	150.7	0.1
8 min. @ 525	21.7	150.8	0.1
8 min. @ 525	21.5	151.5	---

(1) Rice maximum density of 150.9 used to compute voids.

Table 45

Gradation "B" Aggregate Gradations (Old Spring)

Sieve Size Passing	Retained	Compaction Pressure, psi 275			460		
		Percent			Between		
1/2 in.	3/8 in.	10.9	9.1	9.0	8.3	8.0	12.9
3/8 in.	No. 4	35.5	37.1	37.5	36.2	35.4	34.0
No. 4	No. 6	10.6	10.9	10.9	11.0	10.9	10.2
No. 6	No. 8	4.4	3.8	4.3	5.1	5.4	5.0
No. 8	No. 16	7.4	7.4	7.6	7.8	7.5	7.1
No. 16	No. 50	22.5	23.0	23.1	22.6	23.0	21.9
No. 50	No. 100	5.1	5.0	4.8	5.0	5.2	4.8
No. 100	No. 200	0.8	0.8	0.5	0.7	0.9	0.9
No. 200	---	2.8	2.9	2.3	3.3	3.7	3.2
Total retained on No. 6 by wt. of mix		53.6	53.7	54.0	52.2	51.0	53.7

Sieve Size Passing	Retained	Compaction Pressure, psi 595			645		
		Percent			Between		
1/2 in.	3/8 in.	6.5	7.8	7.5	10.7	6.8	7.3
3/8 in.	No. 4	34.6	32.4	31.0	33.8	33.7	34.8
No. 4	No. 6	12.3	11.8	12.4	11.1	10.7	11.6
No. 6	No. 8	5.7	5.5	4.1	5.4	6.4	4.8
No. 8	No. 16	8.3	8.5	8.7	8.2	9.1	8.1
No. 16	No. 50	22.0	21.8	22.4	22.0	21.7	21.9
No. 50	No. 100	3.6	5.4	5.8	3.7	5.4	5.3
No. 100	No. 200	1.8	1.6	2.1	0.3	1.4	1.5
No. 200	--	5.2	5.2	6.0	4.8	4.8	4.7
Total retained on No. 6 by wt. of mix		50.2	48.9	47.8	52.3	48.1	50.5

(Continued)

Table 45 (Continued)

Gradation "B" Aggregate Gradations (Old Spring)

Sieve Size Passing	Sieve Size Retained	Compaction Pressure, psi 5 min. @ 525			7 min. @ 525		
		Percent Between					
1/2 in.	3/8 in.	10.0	8.3	9.1	7.8	8.2	7.5
3/8 in.	No. 4	33.5	34.3	33.8	35.1	33.7	34.1
No. 4	No. 6	11.5	11.7	12.2	11.7	11.8	12.3
No. 6	No. 8	5.2	5.2	5.0	5.3	5.6	5.4
No. 8	No. 16	7.9	8.3	7.6	7.9	7.9	7.9
No. 16	No. 50	22.1	22.8	22.4	22.0	22.4	22.4
No. 50	No. 100	5.0	5.0	5.2	5.0	5.3	5.2
No. 100	No. 200	1.1	1.2	1.5	1.1	1.1	1.2
No. 200	---	3.7	3.2	3.2	4.1	4.0	4.0
Total retained on No. 6 by wt. of mix		51.7	51.0	51.2	51.3	50.5	50.7

Sieve Size Passing	Sieve Size Retained	Compaction Pressure, psi 8 min. @ 525		
		Percent Between		
1/2 in.	3/8 in.	8.4	7.6	8.6
3/8 in.	No. 4	33.4	34.1	33.4
No. 4	No. 6	12.0	12.1	12.3
No. 6	No. 8	5.3	5.4	5.5
No. 8	No. 16	8.1	8.5	8.5
No. 16	No. 50	22.2	22.1	21.7
No. 50	No. 100	5.3	5.0	5.0
No. 100	No. 200	1.3	1.3	1.3
No. 200	---	4.0	3.9	3.7
Total retained on No. 6 by wt. of mix		50.6	50.6	51.0

(Continued)

Table 45 (Continued)

Gradation "B" Aggregate Gradations (Old Spring)

Sieve Size Passing	Sieve Size Retained	Marshall Compacted			7.5 min. @ 595		
		Percent Between					
1/2 in.	3/8 in.	8.9	9.0	9.0	8.3	9.0	7.8
3/8 in.	No. 4	38.4	37.6	37.3	30.2	31.7	32.1
No. 4	No. 6	10.6	10.5	10.9	12.2	11.4	12.4
No. 6	No. 8	4.3	4.6	4.7	5.7	5.5	5.6
No. 8	No. 16	6.6	6.7	6.4	8.6	8.7	8.1
No. 16	No. 50	23.2	23.3	23.6	22.2	21.3	22.2
No. 50	No. 100	4.9	5.1	5.1	5.2	5.3	5.3
No. 100	No. 200	0.7	0.6	0.6	1.7	1.5	1.7
No.200	---	2.4	2.6	2.4	5.9	5.6	4.8
Total retained on No. 6 by wt. of mix		54.4	53.7	53.8	47.7	49.0	49.2

Sieve Size Passing	Sieve Size Retained	10 min. @ 595			4 min. @ 525		
		Percent Between					
1/2 in.	3/8 in.	7.9	8.7	7.4	7.9	8.4	7.7
3/8	No. 4	33.4	33.1	35.0	34.0	34.6	34.0
No. 4	No. 6	10.5	11.7	11.3	12.4	11.8	12.3
No. 6	No. 8	5.9	3.8	5.3	5.2	5.3	5.3
No. 8	No. 16	7.9	8.3	7.9	7.7	7.8	8.0
No. 16	No. 50	22.4	21.6	21.4	23.0	22.4	22.0
No. 50	No. 100	5.3	6.1	5.5	5.1	5.3	5.3
No. 100	No. 200	2.5	1.5	1.7	1.0	1.0	1.2
No. 200	---	4.2	5.2	4.5	3.7	3.4	4.2
Total retained on No. 6 by wt. of mix		48.7	50.3	50.5	51.0	51.5	50.8

Compaction Pressure, psi (both tables)

Table 46

Gradation "C" Marshall Test Results
(Old Spring)

Compaction Pressure, psi	Test Property				
	Marshall Stability, lbs	Marshall Flow, 1/100-in.	Bulk Density, pcf	Percent Mix Voids (1)	Percent Agg. Voids filled (2)
125	1511	16.1	147.8	2.7	67.7
275	2027	29.5	150.3	1.1	84.4
385	2298	21.0	151.3	0.4	93.6
460	2975	24.6	151.6	0.2	96.6
525	3186	19.1	153.3	---	---
595	2506	22.6	153.8	---	---
Marshall Compacted	1371	---	148.9	2.0	74.2
	1275	21.3	147.1	3.2	64.2
	1534	13.3	148.0	2.6	68.8

(1) Rice maximum density of 151.9 used to compute voids.

(2) Design asphalt content of 5.7 percent used to compute percent voids filled with asphalt.

Table 47

Gradation "C" Hveem Test Results
(Old Spring)

Compaction Pressure, psi	Test Property		
	Hveem Stability	Bulk Density, pcf	Percent Mix Voids (1)
125	20.3	146.4	3.6
125	20.6	146.2	3.8
125	19.9	147.8	2.7
275	29.5	149.0	1.9
275	31.2	150.6	0.9
275	31.8	149.3	1.7
385	34.1	150.0	1.3
385	27.4	149.7	1.5
385	39.5	150.3	1.1
460	40.4	151.1	0.5
460	41.5	152.5	---
460	40.6	150.8	0.7
525	---	---	---
525	40.3	152.3	---
525	42.2	153.8	---
595	9.3 (2)	154.3	---
595	6.1 (2)	153.8	---
595	---	---	---
Marshall Compacted	22.6	147.9	2.6
	28.6	147.6	2.8
	23.4	149.2	1.8

(1) Rice maximum density of 151.9 used to compute voids.

(2) Results extrapolated when lateral pressure exceeded maximum dial reading of 200 psi.

Table 48

Gradation "C" Aggregate Gradations
(Old Spring)

Sieve Size Passing	Sieve Size Retained	Compaction Pressure, psi 125			275		
		Percent Between					
1 in.	1/2 in.	27.9	18.5	24.2	23.5	28.0	25.1
1/2 in.	No. 4	38.0	42.8	37.9	39.8	33.5	36.8
No. 4	No. 6	1.0	4.0	3.4	3.6	2.8	3.5
No. 6	No. 8	4.9	3.0	3.1	3.0	2.7	3.1
No. 8	No. 16	6.8	5.5	7.1	6.4	6.8	6.6
No. 16	No. 50	11.2	14.5	12.0	10.7	12.0	11.3
No. 50	No. 100	2.7	2.9	3.0	2.7	2.4	2.8
No. 100	No. 200	2.7	2.9	3.3	2.9	3.7	3.0
No. 200	---	4.8	5.9	6.0	7.4	8.1	7.8
Total retained on No. 6 by wt. of mix		65.0	61.6	61.8	63.1	60.7	61.7

Sieve Size Passing	Sieve Size Retained	Compaction Pressure, psi 385			460		
		Percent Between					
1 in.	1/2 in.	24.1	23.0	25.4	21.9	24.0	27.5
1/2 in.	No. 4	36.8	38.7	36.9	38.2	37.5	34.3
No. 4	No. 6	3.7	3.7	3.6	3.9	3.1	3.4
No. 6	No. 8	3.3	2.9	2.9	3.1	3.1	2.9
No. 8	No. 16	6.9	6.8	7.3	7.1	7.4	7.1
No. 16	No. 50	11.7	11.8	11.9	12.2	12.0	12.0
No. 50	No. 100	3.2	3.2	2.8	2.4	3.1	2.4
No. 100	No. 200	3.5	3.6	3.3	3.9	4.4	3.7
No. 200	---	6.8	6.3	5.9	7.3	5.4	6.7
Total retained on No. 6 by wt. of mix		60.9	61.7	62.2	60.4	60.9	61.5

(Continued)

Table 48 (continued)

Gradation "C" Aggregate Gradations
(Old Spring)

Sieve Size Passing	Retained	Compaction Pressure, psi 525			595		
		Percent Between					
1 in.	1/2 in.	30.3	23.2	21.3	22.9	22.0	22.0
1/2 in.	No. 4	32.2	37.5	39.8	33.1	34.2	34.9
No. 4	No. 6	3.4	4.0	3.6	3.7	3.6	4.4
No. 6	No. 8	2.9	3.2	3.2	3.2	3.3	3.3
No. 8	No. 16	7.1	7.5	7.4	6.4	7.6	7.7
No. 16	No. 50	11.9	12.6	12.3	11.2	12.8	12.9
No. 50	No. 100	3.3	2.9	2.9	8.4	3.6	3.2
No. 100	No. 200	3.9	3.5	3.8	3.7	5.4	3.9
No. 200	---	5.0	5.6	5.7	7.4	7.7	7.7
Total retained on No. 6 by wt. of mix		62.2	61.0	61.0	56.3	56.4	57.8

Sieve Size Passing	Retained	Compaction Pressure, psi Marshall Compacted			
		Percent Between			
1 in.	1/2 in.	30.2	30.2	25.7	
1/2 in.	No. 4	34.6	33.9	38.1	
No. 4	No. 6	2.5	2.9	2.9	
No. 6	No. 8	2.6	2.5	2.6	
No. 8	No. 16	6.6	6.6	7.1	
No. 16	No. 50	11.4	11.6	11.7	
No. 50	No. 100	2.7	2.8	2.7	
No. 100	No. 200	3.0	3.1	3.1	
No. 200	---	6.4	6.4	6.1	
Total retained on No. 6 by wt. of mix		63.5	63.2	62.9	

Table 49

Gradation "D" Marshall Test Results
(Old Spring)

Compaction Pressure, psi	Test Property				
	Marshall Stability, lbs	Marshall Flow, 1/100-in.	Bulk Density, pcf	Percent Mix Voids (1)	Percent Agg. Voids filled (2)
125	1204	12.5	143.6	4.5	57.3
175	1092	11.4	144.1	4.2	59.1
275	1387	11.0	146.1	2.9	67.9
385	1779	9.6	145.5	3.3	65.0
460	2179	10.4	147.3	2.1	74.6
525	2795	12.2	150.0	0.3	95.7
595	2941	10.4	150.0	0.3	95.7
645	3298	19.6	151.1	---	---
Marshall Compacted	875	12.1	143.3	4.7	56.2
	672	8.7	145.0	3.6	62.8
	882	10.5	143.0	4.9	55.2

(1) Rice maximum density of 150.4 used to compute voids.

(2) Design asphalt content of 6.1 percent used to compute percent voids filled with asphalt.

Table 50

Gradation "D" Hveem Test Results (Old Spring)

Compaction Pressure, psi	Test Property		
	Hveem Stability	Bulk Density, pcf	Percent Mix Voids (1)
125	22.9	143.7	4.5
125	---	---	---
125	24.6	144.7	3.8
175	---	---	---
175	27.3	144.1	4.2
175	28.1	145.5	3.3
275	37.6	144.6	3.9
275	---	---	---
275	37.3	147.3	2.1
385	38.5	146.9	2.3
385	37.2	146.6	2.5
385	35.6	147.2	2.1
460	44.8	146.8	2.4
460	39.8	147.0	2.3
460	40.0	148.7	1.1
525	38.7	149.5	0.6
525	41.5	149.3	0.7
525	39.0	148.9	1.0
595	36.8	149.8	0.4
595	43.1	149.8	0.4
595	37.6	150.8	---
645	21.2	151.3	---
645	26.5	150.9	---
645	24.1	151.8	---
Marshall	23.2	142.8	5.1
Compacted	18.5	140.8	6.4
	30.6	142.5	5.3
4 min. @ 595	32.6	150.6	---
4 min. @ 595	34.9	150.0	0.3
4 min. @ 595	31.7	149.9	0.3

(Continued)

Table 50 (continued)

Gradation "D" Hveem Test Results
(Old Spring)

Compaction Pressure, psi	Test Property		
	Hveem Stability	Bulk Density, pcf	Percent Mix Voids (1)
6 min. @ 595	29.8	150.1	0.2
6 min. @ 595	33.5	150.6	---
6 min. @ 595	27.2	151.2	---
8 min. @ 595	24.6	151.6	---
8 min. @ 595	---	---	---
8 min. @ 595	21.4	151.4	---
10 min.@ 595	17.7	152.1	---
10 min.@ 595	11.3	151.9	---
10 min.@ 595	6.8 (2)	151.5	---

(1) Rice maximum density of 150.4 used to compute voids.

(2) Results extrapolated when lateral pressure exceeded maximum dial reading of 200 psi.

Table 51

Gradation "D" Aggregate Gradations
(Old Spring)

Sieve Size Passing	Retained	Compaction Pressure, psi 175			275		
		Percent Between					
3/4 in.	1/2 in.	14.0	17.9	17.6	16.1	17.7	
1/2 in.	No. 4	45.6	43.2	43.6	43.9	41.1	
No. 4	No. 6	8.8	5.2	5.4	6.4	6.8	
No. 6	No. 8		3.5	3.0	3.5	3.7	
No. 8	No. 16	5.8	5.5	5.5	5.4	5.7	
No. 16	No. 50	19.9	19.6	19.6	19.2	19.0	
No. 50	No. 100	3.8	3.5	3.5	3.3	3.6	
No. 100	No. 200	0.7	0.5	0.7	0.9	1.0	
No. 200	---	1.4	1.1	1.1	1.3	1.4	
Total retained on No. 6 by wt. of mix		---	62.3	62.6	62.4	61.6	---

Sieve Size Passing	Retained	Compaction Pressure, psi 385			460		
		Percent Between					
3/4 in.	1/2 in.	18.3	17.2	15.6	15.2	15.6	15.3
1/2 in.	No. 4	40.0	42.8	41.4	41.9	43.5	44.2
No. 4	No. 6	7.4	5.9	8.1	6.2	6.1	5.8
No. 6	No. 8	3.3	3.2	3.3	4.8	3.4	3.5
No. 8	No. 16	5.9	5.6	5.8	5.6	5.7	5.6
No. 16	No. 50	19.1	19.5	19.4	19.3	19.7	19.5
No. 50	No. 100	3.4	3.6	3.6	3.7	3.5	3.6
No. 100	No. 200	1.0	0.9	1.0	1.7	0.9	0.9
No. 200	---	1.6	1.3	1.8	1.6	1.6	1.6
Total retained on No. 6 by wt. of mix		61.7	61.9	61.2	59.5	61.2	61.3

(Continued)

Table 51 (continued)

Gradation "D" Aggregate Gradations
(Old Spring)

Sieve Size Passing	Sieve Size Retained	Compaction Pressure, psi 525			595		
		Percent Between					
3/4 in.	1/2 in.	13.9	13.5	13.3	16.7	12.5	11.9
1/2 in.	No. 4	43.7	43.5	42.7	40.6	44.3	46.0
No. 4	No. 6	6.0	6.1	6.5	6.6	6.3	6.1
No. 6	No. 8	3.5	3.7	3.8	3.7	3.9	3.6
No. 8	No. 16	5.8	6.2	6.6	6.0	6.3	6.0
No. 16	No. 50	19.4	19.8	19.8	19.4	19.6	19.4
No. 50	No. 100	3.9	3.9	3.8	3.6	3.7	3.6
No. 100	No. 200	1.2	1.2	1.2	1.2	1.2	1.2
No. 200	---	2.6	2.1	2.3	2.2	2.2	2.2
Total retained on No. 6 by wt. of mix		59.7	59.3	58.7	60.0	59.3	60.1

Sieve Size Passing	Sieve Size Retained	Compaction Pressure, psi 645			Marshall Compacted		
		Percent Between					
3/4 in.	1/2 in.	11.7	14.5	12.6	16.2	18.2	17.9
1/2 in.	No. 4	40.7	39.8	43.3	44.1	41.4	43.0
No. 4	No. 6	7.2	7.1	7.0	5.8	6.4	5.4
No. 6	No. 8	4.1	3.8	3.5	3.3	3.4	3.4
No. 8	No. 16	7.7	6.8	6.4	5.8	6.7	5.3
No. 16	No. 50	19.3	19.7	19.5	19.7	18.7	20.1
No. 50	No. 100	4.7	4.3	4.4	3.4	3.5	3.3
No. 100	No. 200	1.2	1.1	1.2	0.6	0.6	0.6
No. 200	---	3.4	2.9	2.1	1.1	1.1	1.0
Total retained on No. 6 by wt. of mix		56.0	57.8	58.7	62.1	62.0	62.3

(Continued)

Table 51 (continued)

Gradation "D" Aggregate Gradations
(Old Spring)

Sieve Size Passing	Retained	Compaction Pressure, psi 4 min. @ 595			6 min. @ 595		
		Percent Between					
3/4 in.	1/2 in.	16.2	14.8	12.7	16.0	16.3	16.7
1/2 in.	No. 4	41.5	42.4	44.4	40.2	40.2	38.8
No. 4	No. 6	6.6	6.4	7.1	5.8	7.0	6.7
No. 6	No. 8	3.9	3.8	3.9	4.0	3.6	3.6
No. 8	No. 16	6.3	6.4	6.4	7.3	6.3	7.1
No. 16	No. 50	19.3	19.8	19.7	18.0	19.2	18.5
No. 50	No. 100	3.8	3.8	4.0	4.5	3.8	4.4
No. 100	No. 200	0.8	0.6	0.7	1.2	1.0	1.2
No. 200	---	1.6	2.0	1.1	3.0	2.6	3.0
Total retained on No. 6 by wt. of mix		60.4	59.7	60.3	58.2	59.7	58.4

Sieve Size Passing	Retained	Compaction Pressure, psi 8 min. @ 595			10 min. @ 595		
		Percent Between					
3/4 in.	1/2 in.	10.9	14.3	12.5	20.9	19.0	13.5
1/2 in.	No. 4	45.8	42.4	43.2	36.7	38.9	42.3
No. 4	No. 6	6.8	6.6	6.1	6.3	5.7	7.0
No. 6	No. 8	3.7	3.8	3.6	3.0	3.6	3.8
No. 8	No. 16	6.2	6.8	7.0	6.7	5.8	6.3
No. 16	No. 50	18.8	17.8	19.1	16.9	18.1	18.3
No. 50	No. 100	3.9	3.6	4.1	4.1	3.8	4.0
No. 100	No. 200	1.0	1.1	1.2	1.6	1.3	1.1
No. 200	---	2.9	3.6	3.2	3.8	3.8	3.7
Total retained on No. 6 by wt. of mix		59.7	59.5	58.1	59.3	59.7	59.0

APPENDIX G

RESULTS - OLD COMPACTOR SPRING

APPENDIX G

RESULTS - OLD COMPACTOR SPRING

The summarized results recorded in Table 52 are presented graphically in the series of figures in this section. Marshall Test results are also illustrated from data of Appendix F. Certain aspects and trends observed in these figures are discussed at appropriate points of discusion under the heading of RESULTS in this report.

The compaction and particle orientation produced by use of the low-capacity spring probably are somewhat different than that obtained with the standard spring installed. Thus, the test property results are different for the two springs. Nevertheless, the graphical presentation of the results in this Appendix points out several interesting features about kneading compaction and about the various gradations used in the study.

Even though the recorded compaction pressures for the old spring may not agree with the standard spring, the trends developed, and shown in the following figures, are indicative of what can result in the properties of a mixture by varying the compaction pressure.

Reference is made to the text of the report for further analysis, evaluation, and explanation of the curves presented here.

Table 52

Summary of Hveem Test Results
(old spring)

Gradation	Compaction Pressure, psi	Hveem Stability	Bulk Density, pcf	Percent Mix Voids (a)	Percent Aggregate Retained on No. 6 Sieve by Mix Wt.
A	70	29.6	147.5	3.6	48.6
A	125	34.8	148.2	3.1	49.6
A	225	36.6	150.4	1.5	49.3
A	275	36.8	150.3	1.6	48.7
A	330	35.2	151.4	0.9	49.1
A	385	21.4	152.0	0.5	49.2
A	460	8.6	152.7	0.2	48.0
A	Marshall Compacted	41.0	147.7	3.3	49.4
B	275	34.5	148.2	1.8	53.8
B	460	33.5	149.6	0.9	52.3
B	595	17.4	151.3	---	49.0
B	645	16.4	150.6	0.2	50.3
B	Marshall Compacted	26.8	141.2	6.4	54.0
B	$7\frac{1}{2}$ min, 595	15.5	151.0	---	48.6
B	10 min, 595	11.4	150.8	0.1	49.8
B	4 min, 525	29.9	149.9	0.7	51.1
B	5 min, 525	25.0	150.4	0.4	51.3
B	7 min, 525	24.4	151.0	0.0	50.8
B	8 min, 525	23.0	151.0	0.0	50.7
C	125	20.3	146.8	3.4	62.2
C	275	30.8	149.6	1.5	61.9
C	385	33.7	150.0	1.3	61.6
C	460	40.8	151.5	0.6	60.9
C	525	41.3	153.1	---	61.5
C	595	7.7	154.1	---	56.8
C	Marshall Compacted	24.9	148.2	2.4	63.1
D	125	23.8	144.2	4.1	62.7
D	175	27.7	144.8	3.7	62.5
D	275	37.5	146.0	3.0	62.0
D	385	37.1	146.9	2.3	61.6
D	460	41.5	147.5	1.9	60.7
D	525	39.7	149.2	0.8	59.2
D	595	39.2	150.1	0.4	59.8
D	645	23.9	151.3	---	57.5
D	Marshall Compacted	24.1	142.0	5.6	62.1

(Continued)

Table 52 (Continued)

Summary of Hveem Test Results
(old spring)

Gradation	Compaction Pressure, psi	Hveem Stability	Bulk Density, pcf	Percent Mix Voids (a)	Percent Aggregate Retained on No. 6 Sieve by Mix Wt.
D	4 min, 595	33.1	150.2	0.3	60.1
D	6 min, 595	30.2	150.6	0.2	58.8
D	8 min, 595	23.0	151.5	---	59.1
D	10 min, 595	11.9	151.8	---	59.3

(a) Determined by use of Rice maximum density.

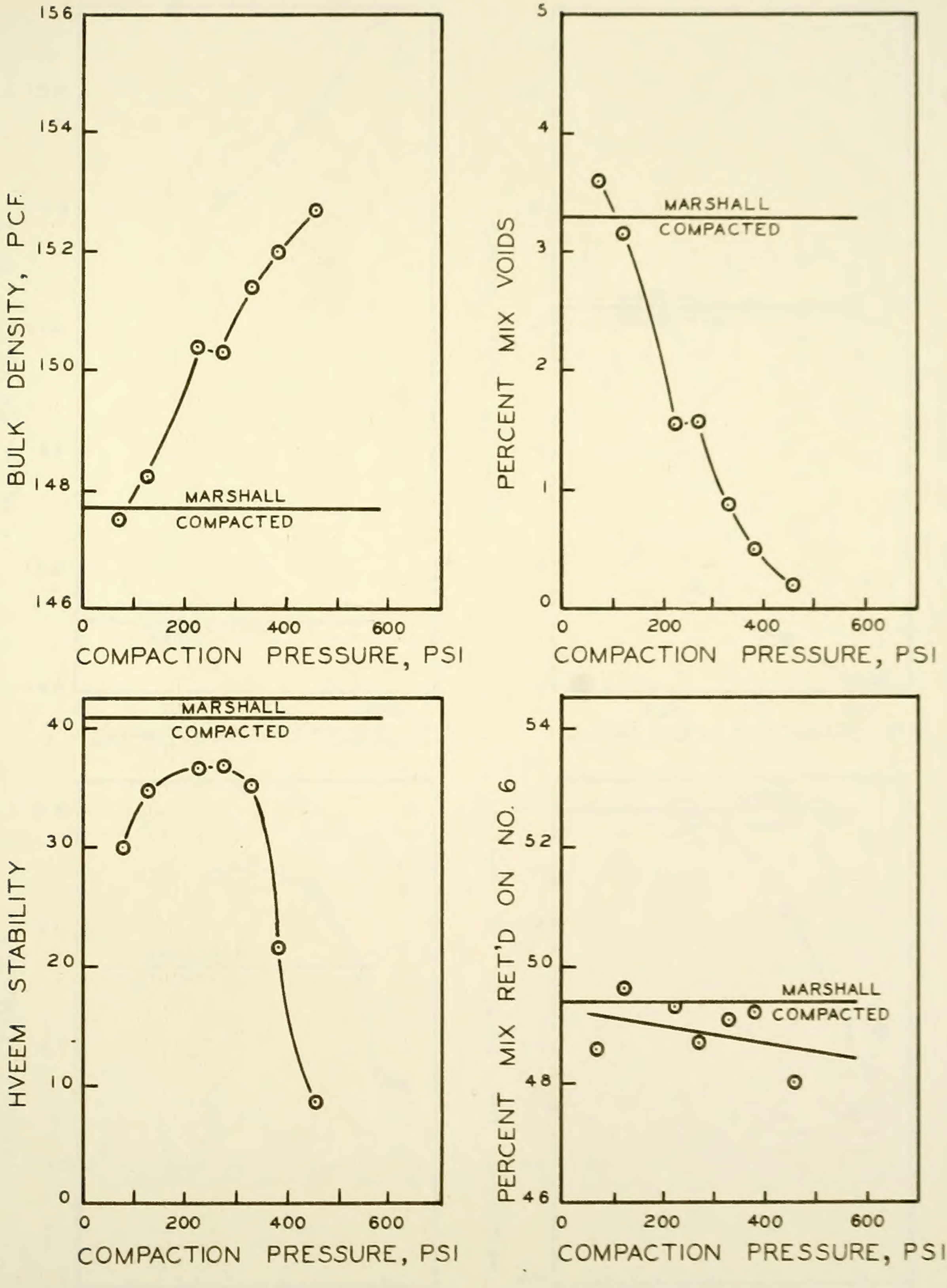


FIG. 40 HVEEM TEST PROPERTY CURVES - GRADATION A
(OLD SPRING)

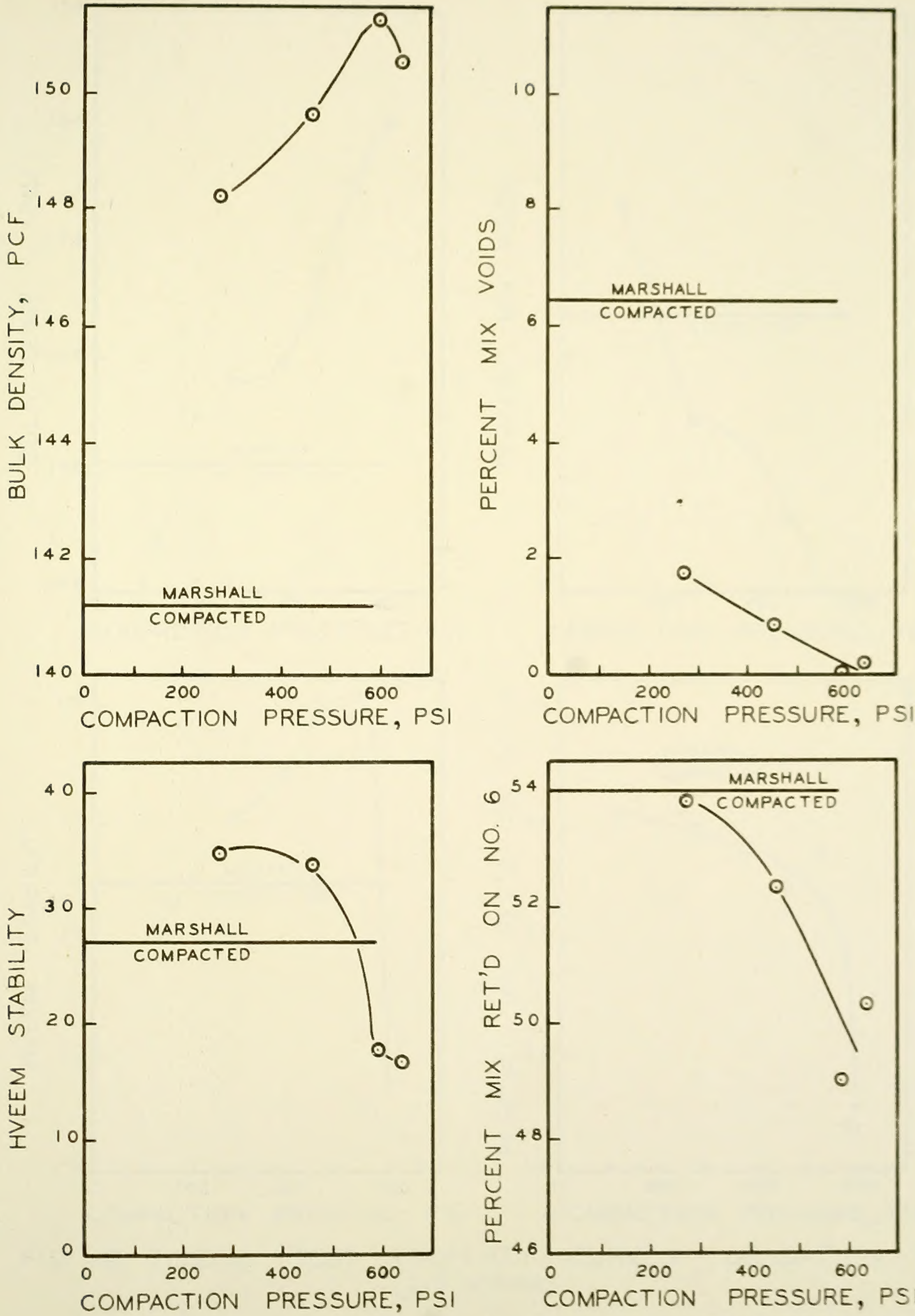


FIG. 41 HVEEM TEST PROPERTY CURVES - GRADATION B
(OLD SPRING)

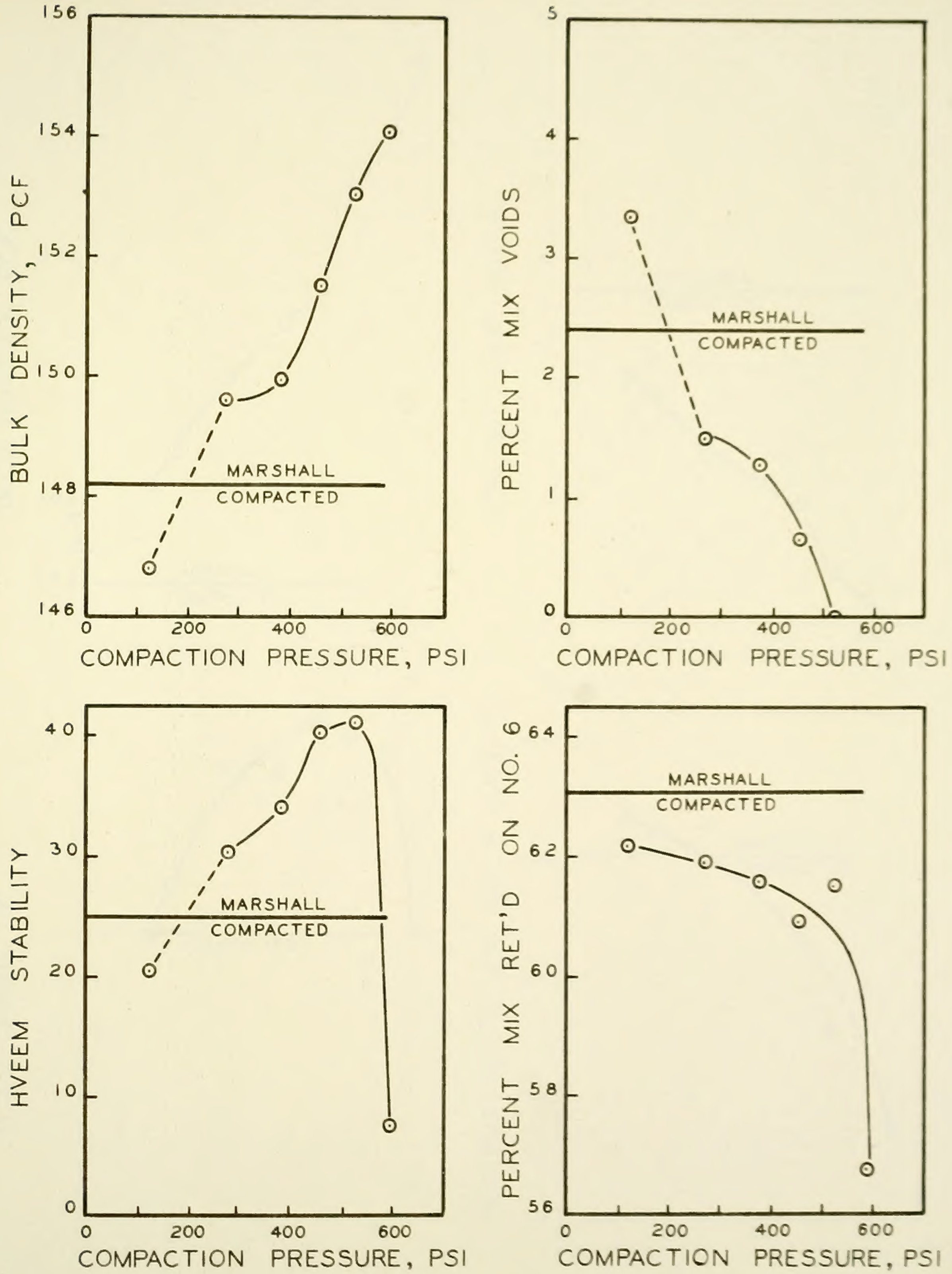


FIG. 42 HVEEM TEST PROPERTY CURVES - GRADATION C
(OLD SPRING)

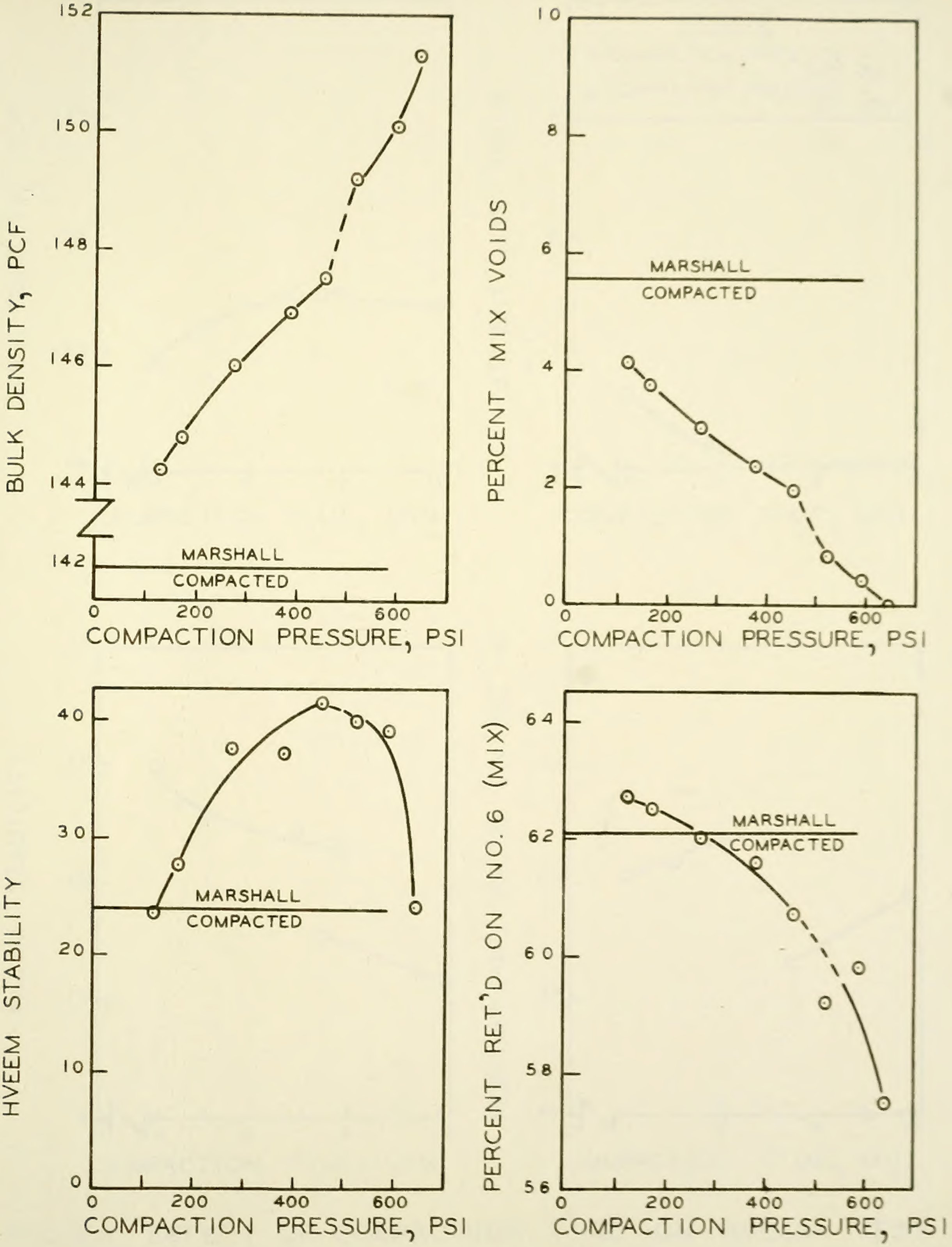


FIG. 43 HVEEM TEST PROPERTY CURVES - GRADATION D
(OLD SPRING)

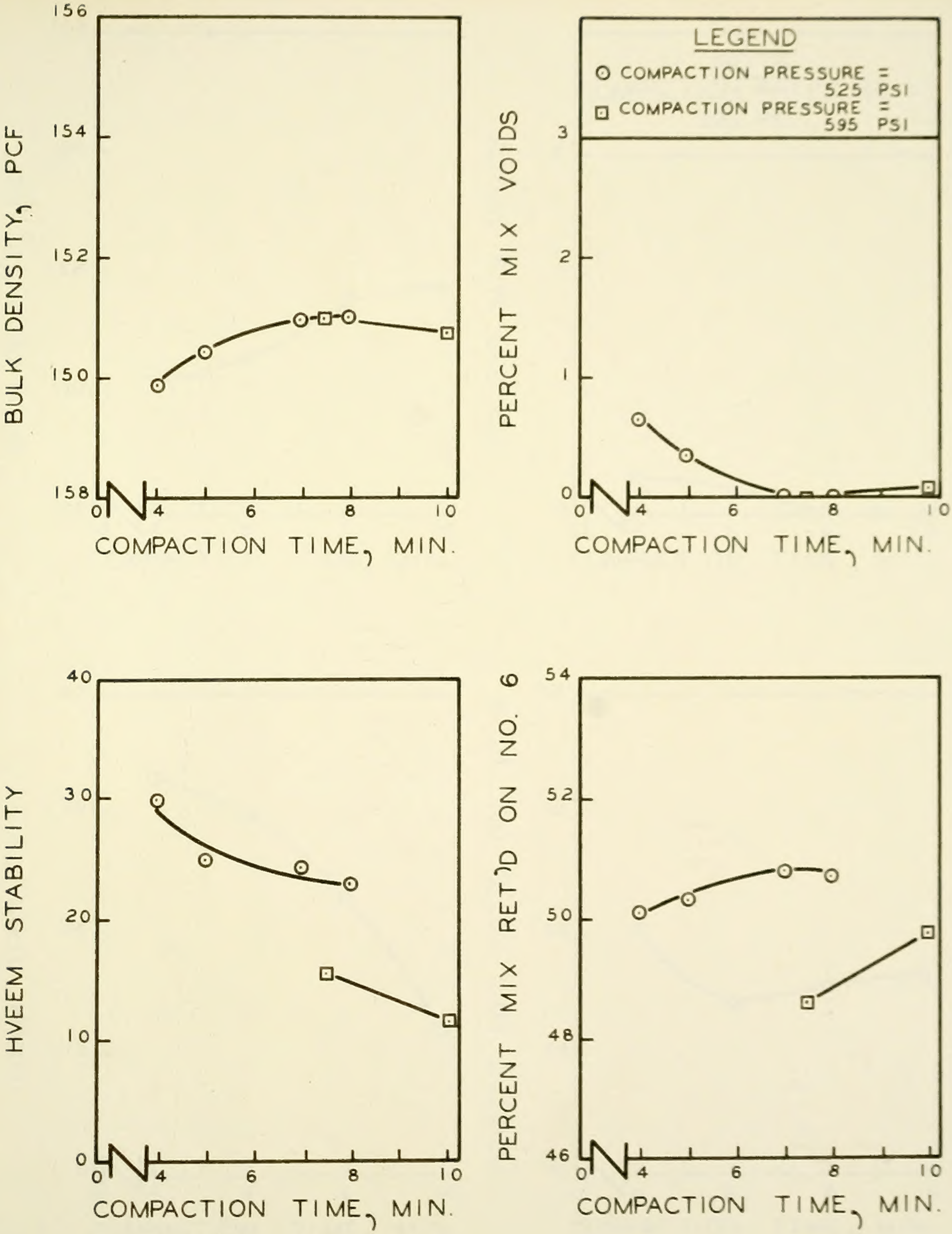


FIG.44 EFFECT OF COMPACTION TIME ON HVEEM TEST PROPERTIES - GRADATION B (OLD SPRING)

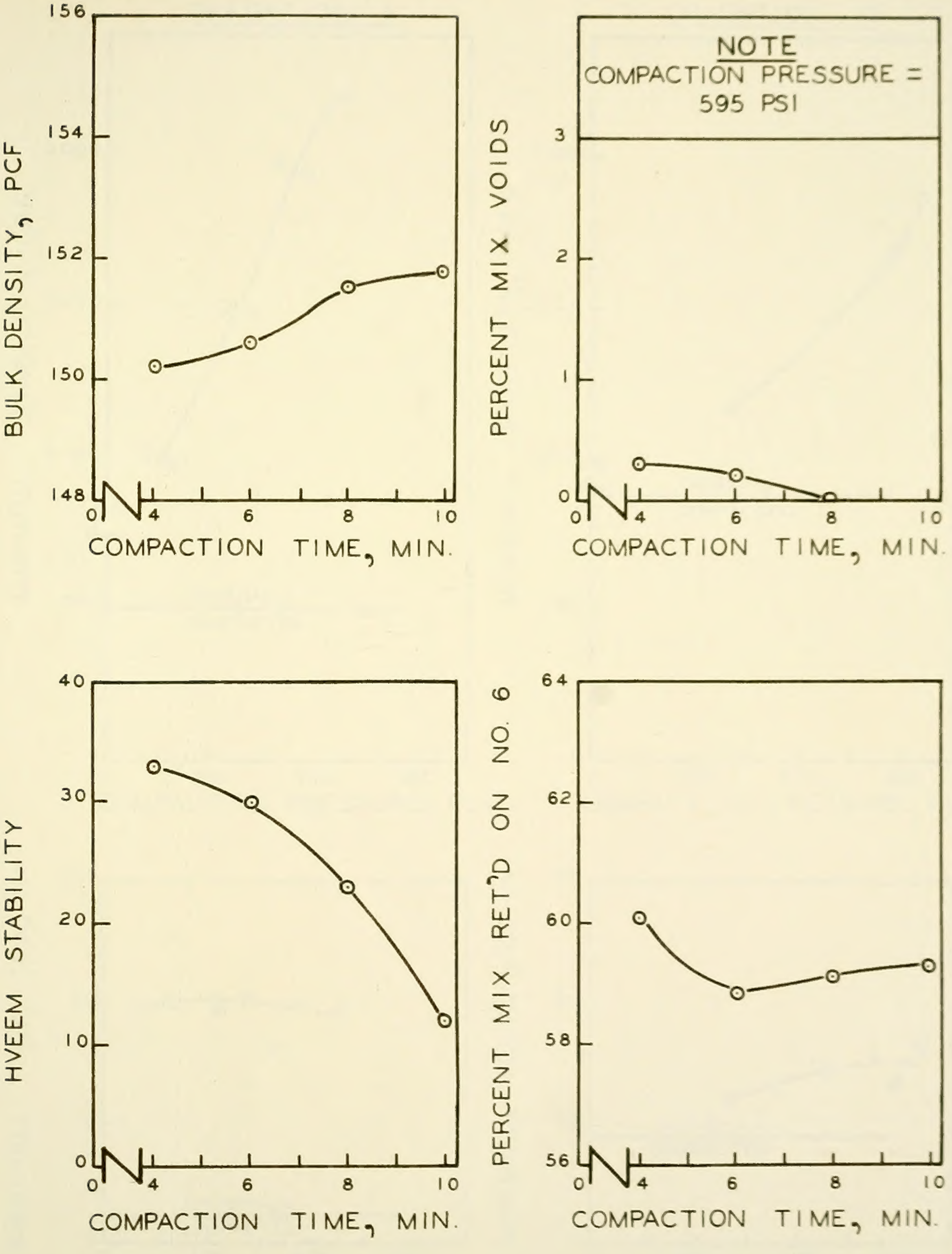


FIG. 45 EFFECT OF COMPACTION TIME ON HVEEM TEST PROPERTIES - GRADATION D (OLD SPRING)

EFFECT OF COMPACTION TIME ON HVEEM TEST PROPERTIES – GRADATION

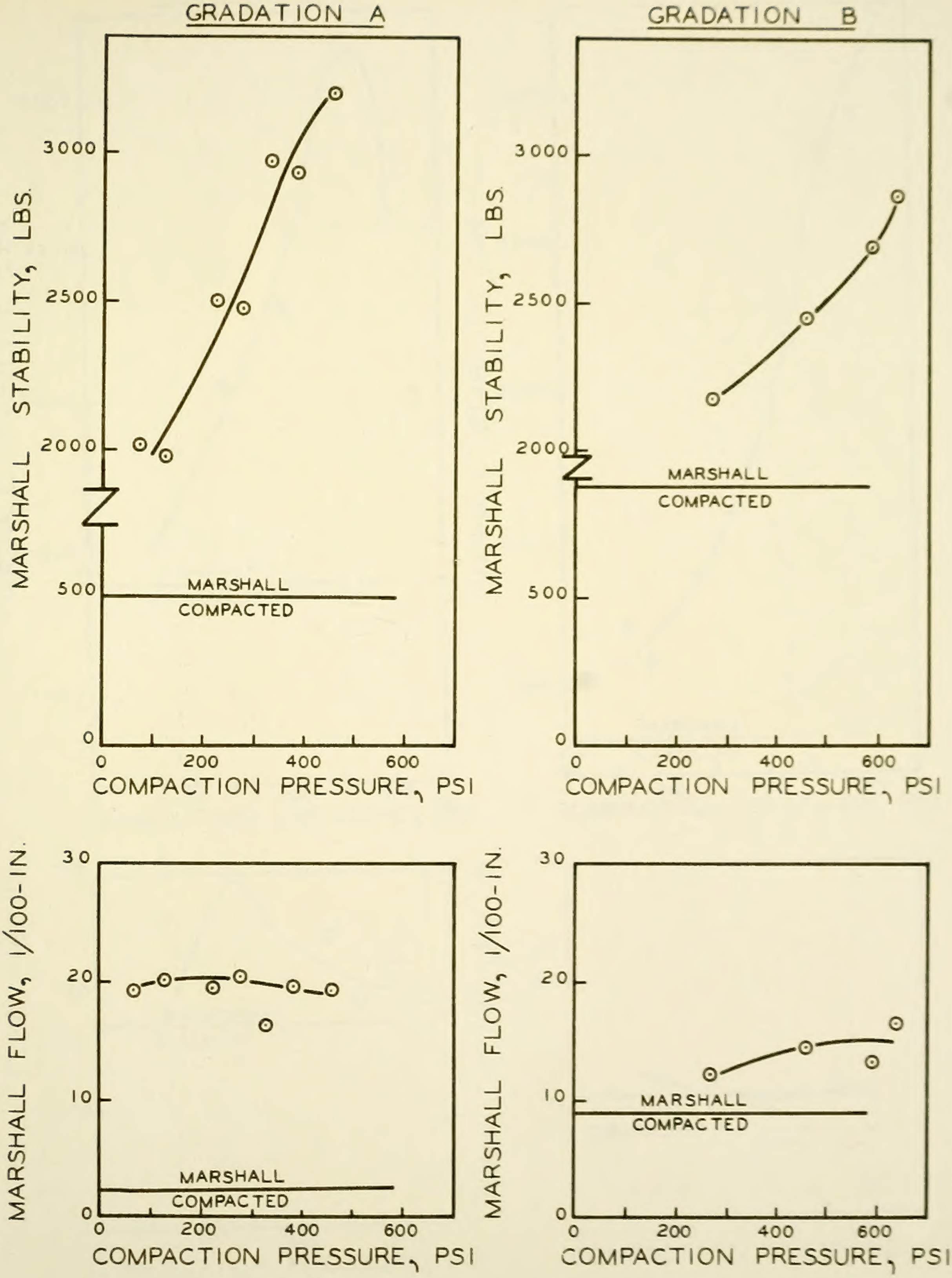


FIG. 46 MARSHALL TEST PROPERTY CURVES - SURFACE (OLD SPRING)

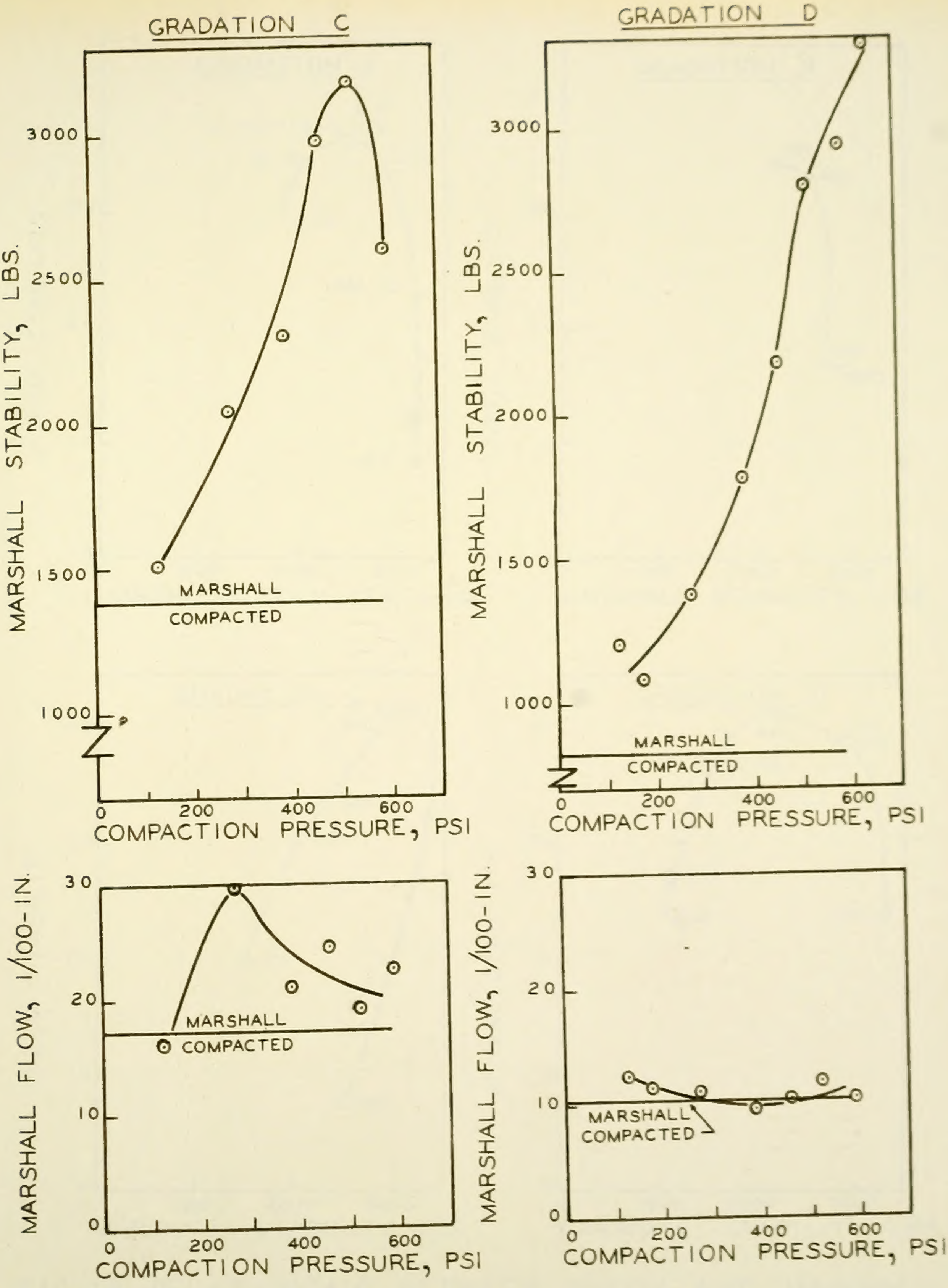


FIG. 47 MARSHALL TEST PROPERTY CURVES - BINDER (OLD SPRING)

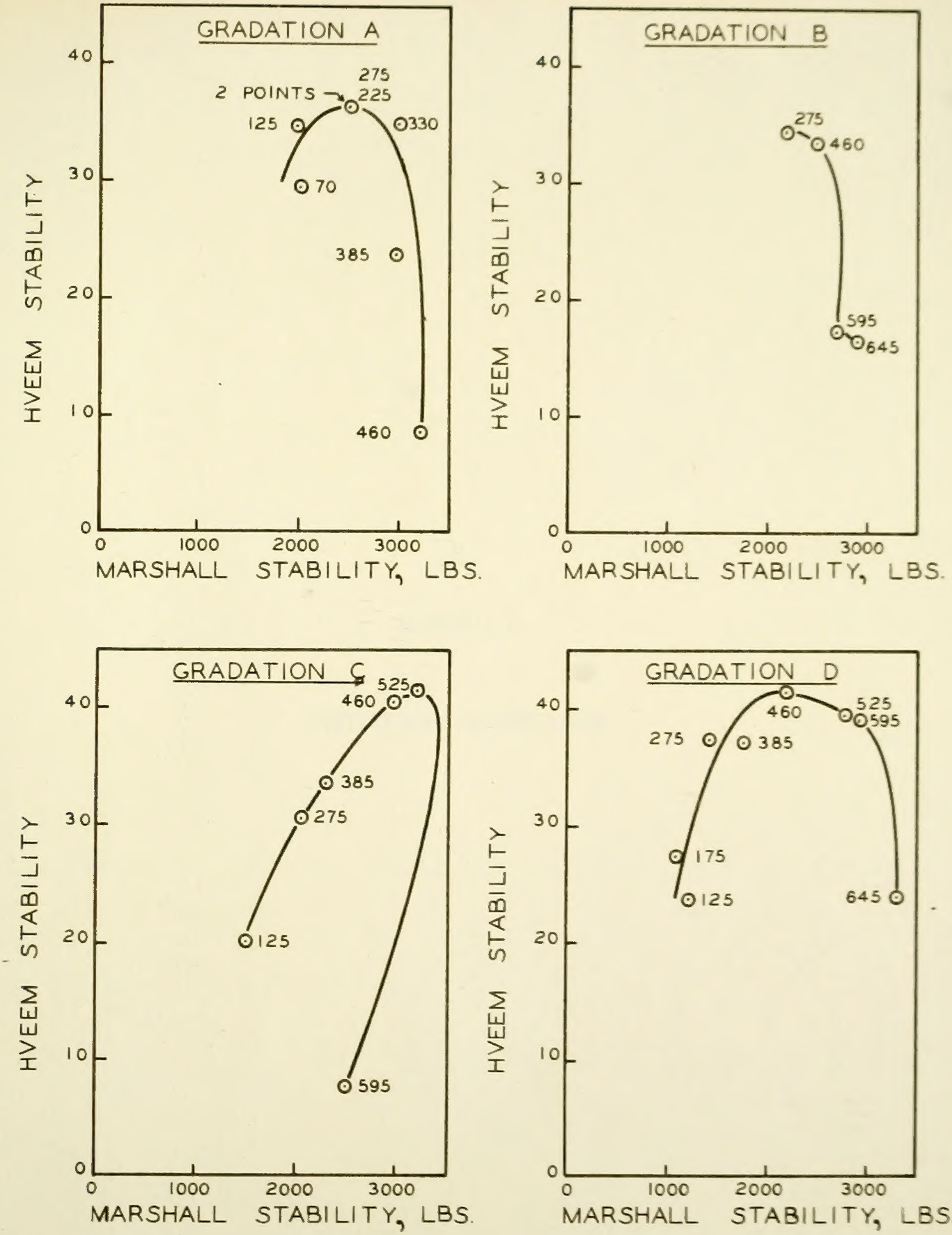


FIG.48 RELATIONSHIP BETWEEN HVEEM AND MARSHALL STABILITY FOR VARYING KNEADING COMPACTION PRESSURE (OLD SPRING)

APPENDIX H

STATISTICAL CALCULATIONS

APPENDIX H

STATISTICAL COMPUTATIONS

A. Effect of the Stabilometer Test on Specimen Density by Comparing Means of Pavement-Sample Densities Before and After Testing.

1. Surface

Bulk Density, pcf		Coded Around 150 pcf	
Before	After	Before	After
153.2	153.4	3.2	3.4
151.5	150.0	1.5	0.0
151.9	151.2	1.9	1.2
152.1	150.8	2.1	0.8
151.6	153.5	1.6	3.5
	152.2		2.2
	151.8		1.8
	153.9		3.9
	152.3		2.3
	151.1		1.1
	153.4		3.4
	152.3		2.3
	151.4		1.4

$\Sigma X_1 = 10.3$

$\Sigma X_2 = 27.3$

$\Sigma X_2^2 = 23.07$ $\qquad$ $\Sigma X_1^2 = 74.49$

$$\bar{X}_2 = \frac{\Sigma X_2}{n_2} = 2.06 \qquad \bar{X}_1 = \frac{\Sigma X_1}{n_1} = 2.10$$

Assumptions: Normal populations, Equal variability within populations, Independent samples

Hypothesis: The two populations have the same mean

Significance Level: Choose $\alpha = 0.05$

$$t_{n_1 + n_2 - 2} = t_{13+5-2} = \frac{\bar{X}_1 - \bar{X}_2 - 0}{\sqrt{\dfrac{\Sigma X_1^2 - \dfrac{(\Sigma X_1)^2}{n_1} + \Sigma X_2^2 - \dfrac{(\Sigma X_2)^2}{n_2}}{n_1 + n_2 - 2}\left(\dfrac{1}{n_1} + \dfrac{1}{n_2}\right)}}$$

$$= \frac{2.10 - 2.06}{\sqrt{\dfrac{74.49 - \frac{745.29}{13} + 23.07 - \frac{106.09}{5}}{13 + 5 - 2}\left(\frac{1}{13} + \frac{1}{5}\right)}}$$

$$t_{16} = 0.07$$

Reject the hypothesis if $t_{16,.05} > 2.12$.

Conclusion: Accept the hypothesis that the two means are equal; the Stabilometer test does not significantly affect the specimen density.

2. Binder

Bulk Density, pcf

Before	After
150.2	153.9
147.2	152.5
150.6	153.1
151.8	153.1
150.9	149.2
	149.0
	147.6
	146.9
	153.0
	150.6
	151.8
	148.4

Coded Around 150 pcf

Before	After
0.2	3.9
-2.8	2.5
0.6	3.1
1.8	3.1
0.9	-0.8
$\Sigma X_2 = 0.7$	-1.0
	-2.4
	-3.1
	3.0
	0.6
	1.8
	-1.6
	$\Sigma X_1 = 9.1$

$\Sigma X_2^2 = 12.29$ $\qquad$ $\Sigma X_1^2 = 72.85$

$$\bar{X}_2 = \frac{\Sigma X_2}{n_2} = 0.14 \qquad \bar{X}_1 = \frac{\Sigma X_1}{n_1} = 0.76$$

Assumptions: Normal populations, Equal variability within populations, Independent samples

Hypothesis: The two populations have the same mean

Significance Level: Choose $\alpha = 0.05$

$$t_{n_1+n_2-2} = t_{12+5-2} = \frac{X_1 - X_2 - 0}{\sqrt{\dfrac{\Sigma X_1^2 - \dfrac{(\Sigma X_1)^2}{n_1} + \Sigma X_2^2 - \dfrac{(\Sigma X_2)^2}{n_2}}{n_1 + n_2 - 2}\left(\dfrac{1}{n_1} + \dfrac{1}{n_2}\right)}}$$

$$= \frac{0.76 - 0.14}{\sqrt{\dfrac{72.85 - \dfrac{82.81}{12} + 12.29 - \dfrac{0.49}{5}}{12 + 5 - 2}\left(\dfrac{1}{12} + \dfrac{1}{5}\right)}}$$

$$t_{15} = 0.51$$

Reject the hypothesis if $t_{15,.05} > 2.13$

Conclusion: Accept the hypothesis that the two means are equal; the Stabilometer test does not significantly affect the specimen density.

B. Comparison of Original and Remolded Density and Stability by Method of Differences

1. <u>Surface Density</u>

Specimen	Corresponding Laboratory Gradation	Bulk Density, pcf Original	Bulk Density, pcf Remolded	Difference
20-0A	B	153.2	153.4	0.2
20-0B	B	151.5	152.4	0.9
20-0C	B	150.8	153.0	2.2
12-0A	B	152.3	154.6	2.3

$\bar{d} = 1.40$ $\quad s_d^2 = 1.047$ $\quad s_d = 1.023$

Assumptions: Normal populations, Matched-pair samples

Hypothesis: There is no difference between the original and remolded results - $\gamma = 0$.

Significance Level: Choose $\alpha = 0.05$

$$t_3 = \frac{(\bar{d} - \gamma_o)\sqrt{k}}{s_d} = \frac{1.40\sqrt{4}}{1.023} = 2.74$$

Reject the hypothesis if $t_{3,.05} > 3.18$

Conclusion: Accept the hypothesis. There is no evidence of a significant difference between the original and remolded results.

2. Binder Density

Specimen	Corresponding Laboratory Gradation	Bulk Density, pcf Original	Bulk Density, pcf Remolded	Difference
20-0A	C	150.2	155.8	5.6
20-0B	C	147.2	154.4	7.2
41-0A	C	150.6	155.4	4.8
41-0B	C	149.2	154.8	5.6
41-0C	C	149.0	153.3	4.3
12-0A	D	151.8	152.0	0.2
12-0B	D	150.9	153.3	2.4
12-0C	D	148.4	155.1	6.7

$\bar{d} = 4.60$ $\qquad s_d^2 = 5.357$ $\qquad s_d = 2.315$

Assumptions: Normal populations
Matched-pair samples

Hypothesis: There is no difference between the original and remolded results - $\gamma = 0$

Significance Level: Choose $\alpha = 0.05$

$$t_7 = \frac{(\bar{d} - \gamma_c)\sqrt{k}}{s_d} = \frac{4.60\sqrt{8}}{2.315} = 5.62$$

Reject the hypothesis if $t_{7,.05} > 2.36$

Conclusion: Reject the hypothesis. There is evidence of a significant difference between the original and remolded results.

3. Surface Stability

Specimen	Corresponding Laboratory Gradation	Hveem Stability Original	Hveem Stability Remolded	Difference
20-0A	B	34.8	16.2	18.6
20-0B	B	23.2	4.4	18.8
20-0C	B	28.9	5.7	23.2
12-0A	B	34.0	17.7	16.3

$\bar{d} = 19.225$ $\qquad s_d^2 = 8.310$ $\qquad s_d = 2.883$

Assumptions: Normal populations
Matched-pair samples.

Hypothesis: There is no difference between the original and remolded results - $\gamma = 0$.

Significance Level: Choose $\alpha = 0.05$.

$$t_3 = \frac{(\bar{d} - \gamma_c)\sqrt{k}}{s_d} = \frac{19.225\sqrt{4}}{2.883} = 13.34$$

Reject the hypothesis if $t_{3,.05} > 3.18$

Conclusion: Reject the hypothesis. There is evidence of a significant difference between the original and remolded results.

4. Binder Stability

Specimen	Corresponding Laboratory Gradations	Hveem Stability Original	Hveem Stability Remolded	Difference
20-0A	C	29.0	25.6	3.4
20-0B	C	22.2	35.8	-13.6
20-0C	C	23.4	24.0	- 0.6
41-0A	C	48.7	40.1	8.6
41-0B	C	27.8	25.0	2.8
41-0C	C	35.4	29.2	6.2
12-0A	D	36.6	20.2	16.4
12-0B	D	15.2	14.5	0.7
12-0C	D	22.4	39.6	-17.2

$\bar{d} = 0.744$ $\qquad s_d^2 = 109.68$ $\qquad s_d = 10.47$

Assumptions: Normal populations
Matched-pair samples

Hypothesis: There is no difference between the original and remolded results - $\gamma = 0$.

Significance Level: Choose $\alpha = 0.05$.

$$t_8 = \frac{(\bar{d} - \gamma_c)\sqrt{k}}{s_d} = \frac{0.744\sqrt{9}}{10.47} = 0.21$$

Reject the hypothesis if $t_{8,.05} > 2.31$.

Conclusion: Accept the hypothesis. There is no evidence of a significant difference between the original and remolded results.